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FLOW OF WATER IN IRRIGATION AND SIMILAR CANALS

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UNITED STATES DEPARTMENT OF AGRICULTURE, WASHINGTON, D. C.

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By FRED C. SCOBEY

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INTRODUCTION

Usually the greatest expense of farming under irrigation, over that of farming in humid regions, lies in the costs of building and operating the irrigation system. Eventually, all or nearly all these costs are borne by the farmer himself in meeting the assessments of his organized irrigation district, mutual company, or water users' association. Obviously, the farmer is seldom equipped to use technical data

[1] Submitted for publication May 9, 1938.

[2] This bulletin supersedes Department Bulletin 194, The Flow of Water in Irrigation Channels.

[3] The author desires to acknowledge indebtedness to the various engineers and managers of irrigation, municipal, and power systems who furnished data or permitted the testing of canals under their charge sometimes aiding in the tests; to the officers of the Bureau of Reclamation who allowed access to original data on tests made by their staff; and to engineers in foreign countries who have sent data concerning their own research. Acknowledgment is also made to the Bureau of Reclamation for photographs illustrating typical conditions to aid in a complete understanding of recommended values for Kutter's n. Obviously, it was not feasible to take photographs of canals tested, without water in them. Tests were made during the height of the season when it was not feasible to turn the water out.

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such as are presented in this bulletin, but he is served by his engineer who is prepared to apply them to bring about economies reflected in those assessments.

This bulletin treats of flowing water in irrigation and similar canals. It is based on field tests made to determine the retardation factors in several formulas applicable to the various conditions found in practice. It is offered for use by engineers in designing, measuring, and operating irrigation, power, municipal, and similar canals, and for courts and attorneys at law interested in cases involving the carrying capacities of open artificial canals.

This bulletin includes the results of many tests made since 1913 under the direction of the author, those made by engineers of the United States Bureau of Reclamation generally following the procedure laid down in Department Bulletin 194 (*55*)[4] and other experiments. The newer parts contain more data for canals conveying clear and muddy waters in shot- and poured-concrete sections, and canals of excessively sinuous line. The text includes comments pertinent to the design and operation of open canals. These comments are the result of more than 25 years of observation on such channels.

Department Bulletin 194 contained such observations as were available on flumes of concrete, metal, and wood. Since then a special study of this type of channel has been made and embodied in a separate publication (*57*), copies of which may be obtained from the Superintendent of Documents, Washington, D. C., for 15 cents each. For that reason all tests on flumes and discussion thereof are omitted from this bulletin. Likewise, the longer discussion of methods and equipment and of basic formulas, found in Department Bulletin 194, are omitted as being fairly well-known by this time. Reference to that publication, which can be found in many public and private engineering libraries, will refresh the mind of the reader who cares to begin at the beginning.

Irrigation systems are designed to supply certain quantities of water to the soil for use by crops. These predetermined quantities of water must be carried in earth, concrete, wood, steel, masonry, cobblestone, or rock-cut channels. Very often one canal will include channels in several of these materials. Obviously, well-kept canals of the smoother materials will convey water with less retardation than is possible by poorly kept channels or those of rougher substance. In order to proportion correctly the size of the canal under any specific conditions anticipated, the extent to which the flow of water will be retarded by the character of the channel and other conditions must be known. This knowledge can come only through actually measuring the flow and all the attendant cross sections and slopes, in canals operating under manifest conditions.

NOTATION AND NOMENCLATURE

NOTATION

Throughout this bulletin, the following symbols are used to designate the same elements. Additional description and definition will be found in Nomenclature.

A—The weighted mean area of the water cross section throughout the length of reach considered, in square feet. (See Nomenclature, weighted values.) Also used for area in the abstract.

[4] Italic numbers in parentheses refer to Literature Cited, p. 75.

a—The area of any particular water cross section, usually given a subscript to identify the location and the corresponding elements.

C—Coefficient in Chezy formula, $V=C\sqrt{RS}$.

d—Depth of water in the channel, in feet, sometimes given a subscript to identify location as d_1, d_2, etc.

d_c—Critical depth. (See Nomenclature.)

d_n—Normal depth. (See Nomenclature.)

E—The elevation of a point on the energy line (*57*), in feet, usually given a subscript to identify location. $E=Z+h=k+d+h$.

g—The gravitational constant $=32.2$ in English measures.

H—Energy content$=d+h$. Bakhmeteff's "specific energy" (*3*).

H_{min}—Minimum energy content$=d_c+h_c$.

h—Velocity head, assumed$=\frac{v^2}{2g}$, in feet;[5] the drop in elevation of the water surface necessary to generate the velocity under consideration.

h_c—Velocity head for Bélanger's critical velocity, v_c, in feet.

h_f—Friction loss. The fall in the energy line, through the length of reach considered, in feet. The difference in the values of E at the two ends of the reach. For idealized steady uniform flow only, the fall in the energy line, in the water surface and in the channel bed are alike. The loss due to all hydraulic roughness rather than perimeter-contact friction only.

k—Elevation of canal bed above datum, in feet.

L—Length of reach considered, measured along the bed slope, in feet.

n—Coefficient of hydraulic roughness in Kutter's formula. (An indicator of the nature of the channel.)

n'—Coefficient of hydraulic roughness in Manning's formula.

P—Wetted perimeter in the abstract.

p—Wetted perimeter, in feet, at a particular cross section, from which $\frac{a}{p}=r$.

Q—The amount of flow, in cubic feet per second, under consideration. Design-Q', the flow used as maximum capacity in the design of a canal or other structure.

R—Mean value of hydraulic radius (r_1, r_2, r_3, etc.) through the reach considered, in feet, for computation of roughness coefficients. Also R used for hydraulic radius or hydraulic mean depth, in the abstract.

r—Hydraulic radius at a particular cross section, in feet, usually given a subscript to identify location as $r_1=\frac{a_1}{p_1}$.

S—The slope of the energy line (E line); always downward in direction of flow. The slope factor in flow formulas such as Kutter's. $S=\frac{h_f}{L}$ in feet per foot. Except in uniform flow, S is not the slope of the water surface. In idealized flow, it is parallel to and hence equal to slope of the surface and that of the channel bed.

s—Slope of the bed of the canal, usually downward. In design, s usually is computed as parallel to E line and to water surface for an assumed normal surface at capacity flow.

T—Width of water surface at section under consideration, in feet.

V—The mean velocity of the water through the reach under consideration, in feet per second. Approximates $\frac{Q}{A}$ if cross sections are taken often enough so that mean area A closely approximates the mean value of a great many local areas (a_1, a_2, a_3, etc.). In chutes, the swell of the water due to entrainment of air may annul the continuity equation $V=\frac{Q}{A}$. (See p. 53.) V is also used for velocity in the abstract.

v—The average velocity of water in a local cross section, usually given a subscript to identify location, as $v_1=\frac{Q}{a_1}$.

Z—Elevation of the water surface above datum, in feet, usually given a subscript to identify location as Z_1, Z_2, etc.

WS—Water surface; usually sloping downward. May remain level though flowing fairly rapidly throughout length, L, say 1,000 feet. This emphasizes the

[5] This is the usual interpretation of velocity head. Strictly speaking, it should be the mean value of the velocity heads for all the elements of flow across the section, rather than the velocity head for the mean velocity across the section. The true value may be 15 percent or more in excess of h as given here.

fact that energy slope, S, is effective, not water slope. May rise abruptly, through the hydraulic jump. May rise and fall intermittently—above and below critical depth, either with or without the jump, for flow near critical depth in a uniform channel.

NOMENCLATURE

In this bulletin certain words and phrases have special meanings not found in the ordinary dictionary.

Canal: In the irrigated West, "canal" is the name usually applied to an irrigation channel and "ditch" to a drainage channel, even relatively small channels. This usage is followed in the irrigation and drainage reports of the United States Bureau of the Census, especially as applying to main canals. Smaller channels leading from these are termed "laterals" and a still smaller distributary a "farmer's lateral" or a "head ditch." Seldom is the word "canal" applied to any drainage channel. Canals and laterals are usually "designed." The digging of smaller farm laterals or head ditches is usually dependent on experience and the implements at hand.

Critical depth: In design and operation, critical velocity (attributed to Bélanger) and the depth at which this velocity holds are of importance. For any given section, quantity of flow, Q, and total energy, H, there are two depths of water for which $d+h$ are identical. These are called the "alternate stages." At critical depth, these two stages merge. For any other such value of H, there are two other depths that are also conjugate (*57, p. 75.*) Water at less than critical depth is flowing at shooting velocities and the channel usually becomes a "chute." If the depth is greater than the critical, velocities are streaming, subject to both the backwater curve and the drop-down curve. Shooting velocities are not subject to long backwater or drop-down curves, as ordinarilly considered. They do exist under the condition of accelerating flow, from critical velocity at the top of a chute to a much faster velocity that is the normal for the chute under consideration. Problems involving conditions mentioned in this paragraph are found in the design of spillway channels protecting reservoirs and in steep channels (chutes) lowering a canal from one general elevation to another of much less elevation.

Energy slope: The rate of fall of the energy line. $S=\frac{h_f}{L}$.

Energy line: The energy grade line. The locus of Bernoulli's summation, considering losses; hence, the E line. It should not be confused with water surface. For tapered flow the energy line is taken as straight for the reach being considered. If the taper is caused by checked water, the depth increases, velocity decreases, and the E line is slightly concave upward. If the velocity increases the E line is slightly convex upward.

Hydraulic roughness: Cause of loss of head in flowing water; includes influence of channel surface and alinement, mosses, silts, sands and all the other losses that go to make hydraulic roughness different from mere channel surface friction.

Normal depth: The depth of water at normal flow d_n (see). The idealized depth resulting from computations for uniform flow. Some writers prefer "neutral depth" to avoid any confusion with "normal" meaning "at right angles to." For flow down steep chutes, depth is measured normal (at right angles) to the slope of the chute rather than vertically. Only in this connection can confusion arise in the use of normal depth as defined above.

Normal flow: Uniform flow in a uniform channel, satisfying the solution for a flow formula, such as Kutter's. Under this condition, the bed slope, the water surface, and the energy line are parallel. Though useful in design, such uniform flow is seldom found in field experiments. Long, straight channels of uniform shape and uniform surface would develop this idealized flow. It should not be taken for granted in any field tests for values of n. Some writers prefer "neutral flow" to normal flow.

Regime flow: Established condition with no scour and no silting in a canal built up of the silts conveyed in the water (*32, 35, 37.*)

Shooting and streaming flows: See Critical depth.

Weighted values: Throughout this bulletin values of local elements, such as a, r, etc., are weighted in the determination of corresponding mean values, A, R, etc. in accordance with the length of reach each local element influences.

CAPACITY FORMULAS AND ATTENDANT EMPIRICAL DATA

Preceding a description of the methods, equipment, and results of field observation relating to the capacity of canals under various conditions, the formulas used in design of canals from the capacity standpoint must be shown in order to make clear the various hydraulic elements entering them and to disclose how the values of these elements are developed from field measurements.[6]

Although the conveyance of water in artificial channels was one of the earliest of engineering achievements, as yet no rational capacity formula (as substituting for an empirical formula) has been accepted. Therefore, the value of any recognized formula lies in the amount of empirical data, developed by the best of field experiments, that can be placed at the disposal of the engineer seeking the solution of a problem of flow. In all the formulas offered in the following pages, all elements are of assured measurable dimensions with the exception of the coefficient of roughness. Evaluation of this element, variable over a wide range, is the goal of experiments such as those listed in this bulletin.

While seldom attained in canals under operation conditions, uniform, normal flow must be assumed in design, except in special locations where variations can be fully anticipated and provided for.

EMPIRICAL EQUATIONS FOR FLOW IN UNIFORM CHANNELS

Current best practice still warrants the use of the Chezy formula, viz:

$$V=C\sqrt{RS} \tag{1}$$

with the values of C developed from careful field tests, in terms of the Ganguillet-Kutter formula (hereafter referred to as the Kutter formula) (*23*),

thus:

$$V=\left\{\frac{\frac{1.811}{n}+41.66+\frac{0.00281}{S}}{1+\left(41.66+\frac{0.00281}{S}\right)\frac{n}{\sqrt{R}}}\right\}\sqrt{RS} \tag{2}$$

All the elements in this formula can be found from simple engineering dimensions, except the value of n. It is the so-called coefficient of roughness, and was originally developed from 81 series of gagings on open channels.

Many able hydraulicians of the past decade or two have advocated the use of the Manning formula as superior to that of Kutter.[7] The reader desiring to pursue this idea is referred to the following authori-

[6] It is assumed that the reader is familiar with the essentials of the development of the Ganguillet-Kutter formula in 1869, to evaluate the element C in the Chezy formula of 1775, since it had been found that C was not a constant as assumed in the work of Chezy. The complete history of the development of "Kutter's formula as it is called in the United States is given by Ganguillet and Kutter (*23*), and in some of the more elaborate works on hydraulics. Comparison with other formulas is made by Houk (*30*) and Gaby (*22, p. 899.*)

[7] Some readers will question the use of the Kutter formula in this revision instead of the Manning formula. With all its known faults, the Kutter formula is still used very generally for the design of canals and similar conduits (flumes, nonpressure tunnels and the like) by organizations most familiar with both formulas, such as the U. S. Bureau of Reclamation, most of the organized irrigation districts of the United States, the various agencies building and operating canals in India and Argentina (*4*), Peru, and Chile (*46*), South Africa, and Switzerland (*62*). In Italy (*53, 54, 68*) recent research work has been carried on in the terms of Bazin (*17*) and Ganguillet-Kutter (commonly referred to in this country and in this publication as "Kutter"), with no mention of Manning. It has come to be recognized that the so-called slope term, i. e., $\frac{0.00281}{S}$ was developed to make the original Kutter formula conform to the Humphreys and Abbot measurements on the Mississippi River, and now those measurements of slope are generally discredited. However, the slope term has but small influence, except in the very flat gradients, say below $S=0.0003$, which are used only for very large canals.

ties: King (*33, 34*), Horton (*36*), Lindquist (*38, 39*), Leach (*36*), Houk (*31*), Parker (*47*), Blanchard (*2*), and Chivvis and Monteith (*9*).

For comparison the field data in table 1 have been used to develop Manning's n' as well as Kutter's n.

Manning's formula [8] is

$$V=\frac{1.486}{n'}R^{0.67}S^{0.50} \tag{3}$$

Other formulas that have been given consideration in both Europe and, to some extent, in the United States are those of Strickler (*61*), Lindquist (*38*), Matakiewicz (*42*), and the Bazin formula of 1897 (*34, p. 256*). Quite often a problem arises as to the proper equivalent value of n or n' to use for canals with bottom and sides of entirely different characteristics but whose individual values of n or n' are known within narrow limits. For instance, the determination of the equivalent value of n' to use for a concrete-lined bottom (say, $n=0.014$) in a rock cut (pl. 21, *A*) where the sides have been somewhat improved by the use of shot concrete (say $n=0.025$). In 1933 Horton (*28, 29*) offered a formula and a graphical method for the solution of this problem. Letting n'_b and n'_s represent the values of n' (Manning) for the bottom and for the sides respectively, and letting z=the ratio of the length of one side of the cross section to the width of the bottom, then the equivalent value of

$$n'=\left(\frac{n'^{3/2}_b+2zn'^{3/2}_s}{1+2z}\right)^{2/3} \tag{4}$$

NECESSARY FIELD DATA FOR VALUES OF n

As previously stated, n is the one element not easily and assuredly determined in office estimates of canal design. Therefore the field data must be obtained with a view to solving the equation in Kutter's formula, the value of n being the desired answer.

For the sake of brevity in computation, in formula (2) let $B=k+\frac{m}{S}$ where k is 41.66 and m is 0.00281, and let $e=1.811$, while C is the Chezy coefficient, equal to $\frac{V}{\sqrt{RS}}$, then

$$n=\sqrt{\frac{e\sqrt{R}}{BC}+\frac{1}{4}\left(\frac{C-B}{BC}\right)^2R}-\frac{1}{2}\left(\frac{C-B}{BC}\right)\sqrt{R} \tag{5}$$

[8] A short discussion appears warranted to show the degree of conformity between Kutter's n and Manning's n'. The use of identical values of n and n' do not yield the same results over the range of conditions found in canals considered in this bulletin. For the lined canals and others of moderate size, with values of say $n=0.012$ to 0.020, practically identical results are obtained. Through this range, too, the designer is most likely to assume the value of n that will actually hold in the constructed channel. When the two formulas are applied to very large canals or channels of excessive hydraulic roughness, the divergence between, results is greatest. Likewise, in these zones the designer is less certain of hitting upon the right value for the roughness coefficient. For instance, irrigation canals seldom have values of R less than 0.4, and such channels, lined with smooth concrete under excellent conditions, would have a value of $n=0.013$ with a slope of say 0.001. Solution for the velocity, V, by either Kutter or Manning formula will yield approximately the same results for the same value of n and n'. On the other hand, the great All-American Canal, now (1937) being constructed to serve Imperial Valley in California, has the following hydraulic properties at the head end: $Q'=15{,}155$ second-feet; $R=16.6$; $S=0.0000528$ and $V=3.75$ for a design value of Kutter's $n=0.020$. To obtain the same computed value of V by the Manning formula would have required the use of $n'=0.0187$. Likewise, for the rock-cut section of this canal, $Q'=10{,}155$ second-feet; $S=0.000538$; $R=14.19$; and $V=6.0$ feet per second for a design value of $n=0.035$ for the theoretical section. To obtain the same value of V by the Manning formula, using the same values for the other hydraulic elements, would require that of $n'=0.0328$. For a comparison of values of n and n' applicable to a wide range of conditions, the reader is referred to King's hydraulic tables (*34, table 92*).

To solve Manning's n', transpose formula (3)

$$n'=\frac{1.486}{V}R^{0.67}S^{0.50} \tag{6}$$

For both the Kutter and Manning formulas field determinations result in known values for the same elements:

1. The mean velocity of the water prism, V
2. The mean hydraulic radius, R.
3. The effective slope (that of the energy gradient), S.

From these elements, the value of n or other coefficient is determined.

None of the elements above is found by single direct measurements in the field. The field measurements cover the following items:

1. Measurements of a definite length of reach, L. A length of about 1,000 feet is excellent. For large canals on flat gradients a longer reach is desirable.
2. Careful current meter, weir, or other measurements that will yield the discharge, Q. This discharge should hold steadily throughout the field measurements for all elements.
3. Measurements that will yield the cross-sectional area and wet perimeter of the water prism at the two ends of the reach and as many intermediate locations as are feasible.
4. The actual or assumed elevation of the water surface at one end of the test reach and measurements to all feasible accuracy of the corresponding elevation at the other end.

The other field data to be taken in order to make the resulting value of n fully comprehensible comprise a careful description of the material forming the containing channel, including such aquatic and larval growths as affect the flow of the water, and the influences of all structures in the canal and all changes in alinement throughout the reach tested. This general description not only should cover the reach tested, but should also extend upstream and downstream for sufficient distances to include anything influencing the flow within the reach.

Temperatures of the air and water may be taken, but it is doubtful if any deductions may be made as to the direct influence of the various temperatures on the flow of water in the usual more or less irregular channel.

SCOPE OF EXPERIMENTS

Tests were made on channels in Arizona, California, Colorado, Idaho, Louisiana, Montana, Nebraska, Nevada, Oregon, Texas, Utah, and Washington. These channels ranged in size from small ditches carrying less than 1 second-foot up to canals carrying more than 2,600 second-feet. The materials comprise concrete, earth, rubble masonry, cobblestones, wood, and special combinations. Velocities encountered extended up to about 30 feet per second. From other sources the author has obtained the data for additional tests, especially at very high velocities where in his opinion there was not sufficient evidence in results of his own experiments from which to draw conclusions.

In several cases it was possible to get data covering several tests on exactly the same reach of channel, with varying discharges of water, to indicate the trend in values of n with changes in depth.

EQUIPMENT AND METHODS EMPLOYED FOR COLLECTING FIELD DATA [9]

Linear measurements.—These were made with engineer's tapes 25, 50, or 100 feet long.

Leveling.—A sensitive 18-inch wye level and the best of Philadephia-type rods were used. (Datum can be assumed if not known.) Sights were equalized and made short enough so that the rod could be read directly, without use of the target. Check levels closed the circuit. Levels were re-run until it was certain no appreciable error had been made.

Discharge measurements.—These were usually made with current meters. For the measurements recorded in Department Bulletin 194 (*55*), the Price cup meter was used exclusively. For measurements made during the past 10 years, the European type with horizontal axis (Hoff meter) was mostly used. Meters were always rated just before or just after any extended series of tests; however, ratings do not change materially when a meter is handled carefully in measuring canal flows, this service being much less violent than river gaging. Usually, but not necessarily, the meter station was one of the section locations along the length of reach tested. For tests on narrow canals, footwalks or planks were used. For large canals a portable equipment trunk was devised. This trunk, when hung by sheave wheels from a cable anchored at the ends by portable steel pins, became a gaging car (pls. 11, *B* and 16, *B*). The gaging car trunk packed with the testing equipment, was shipped as baggage from place to place. The cover was without hinges but had duplicate lock-and-clip appliances on both sides so that it could be lifted off while the trunk was used as a gaging car. When in experimental use, the car was suspended from and traveled on 130 feet of fine ⅝-inch steel haulage cable.

When it was necessary to make a current-meter measurement of discharge in a canal where grass or moss might clog the meter and make the measurement inaccurate, or where the bottom was slightly uneven, the sides and bottom were neatly trimmed by means of a sharp short-handled hoe.

The steps taken in obtaining the field data and making the office computations, in most of the experiments made by the engineers of the Bureau of Agricultural Engineering, may be outlined as follows:

IN THE FIELD

1. Arrange for a steady flow of water in the canal throughout the test and, if possible, have that flow undisturbed over night or longer, so that a regimen of flow has become established before the test experiments are begun.

2. Select the reach to be tested, L. This need not be straight, but should be typical of a definite category and long enough to develop a definite fall. Most canals have an appreciable amount of curvature, so it is desirable to include moderate curvature in the test reach. Often the test reach can be selected near the gaging station used for routine measurements of the canal flow. That station may even be the upper or lower end of the reach; often, however, such a station has special size and shape and should not be included in the test reach.

3. Start the hydrographer on the measurement of the discharge, Q, by the best method available (*55*, p. 12). This is usually a direct-field measurement by current meter or otherwise. The elaborate meter gagings from which data in this bulletin were developed took from 3 to 5 hours. The meter was held at enough points in each "vertical" to develop the vertical velocity curves. The width of the canal was divided into 10 to 20 verticals—more than usual. Most of the gagings were thus based on velocities at from 50 to 125 or more points in the cross section. Check measurements were usually made by the integration method. The discharge gaging was made while the other data were being developed by the rest of the field party.

4. Carefully chain the length of the test reach, L, leaving the pins or stakes for use in the subsequent operations.

5. Choose as many cross-section locations as are feasible. On a reach, say 1,000 feet long, the locations at the ends of the reach are important and should certainly be taken. Others desirable are the midpoint and quartering points. To develop the procedure, assume these five locations are chosen and number them (i. e., locations 1, 2, 3, 4, 5). For all weighting, equidistant locations have a multiplier of 1.0 for each of the two ends and of 2.0 for the intermediate locations.

6. For each location take measurements that can be developed into cross-sectional shape, area, and wet perimeter of the water prism, a_1, a_2, etc., and p_1, p_2, etc.

7. By most careful use of the level and rod determine the relative elevations of the water surface at the two ends of the reach at least, Z_1 and Z_5. In the experiments conducted by the author these ends were usually marked by nail heads flush with the water surface and set with a hook-gage device used in a stilling well. These nails could be used directly as points for the level rod. The stilling well was a simple tin box about 3 inches in diameter with a small hole in the bottom permitting it to extend below the nail. Water would rise in this box, through the hole, in such a quiet condition that the nail could be driven with the hook-gage device to all the accuracy required. In concrete-lined canals the nails could be driven vertically in cracks usually found at expansion joints. If the lining was without cracks, a small hole was made with a steel punch and the nail set with the stilling box and hook device as before described.

Sometimes the leveling was done between well-set stakes near the edge of the water at the ends of the reach, and by secondary measurements with a hook gage the relationship between the stake-bench mark and the water surface was determined. This process (or its equivalent) is best where a series of tests is to be made, perhaps over a long period. The leveling should be as nearly precise as possible, especially on large canals of gentle slope where a few thousandths of a foot become a large percentage of the friction loss. Erroneous empirical values of n can generally be traced to false values or interpretation of S

IN THE OFFICE

1. From the current-meter notes, depth-velocity curves for the various verticals are platted and, by planimeter, the mean velocity in each vertical is determined from the resulting vertical velocity curve. This velocity is multiplied by the area of the vertical strip to which it applies, to determine the local quantities of flow in each strip; and the integrated value of the local flows determines Q, the total discharge

[9] For the sake of brevity detail descriptions given in Department Bulletin 194 (*55*) are omitted here

in the canal. The cross sections for each location are now platted on fairly large scale. For uneven sections the areas a_1, a_2, etc., may be planimetered while for lined canals they may be computed and, for a weighted value,

$$A=\frac{a_1+2a_2+2a_3+2a_4+a_5}{8} \tag{7}$$

The wetted perimeters, p_1, p_2, etc., may be determined by computations or by stepping around the perimeter with dividers. Local values of the hydraulic radius are next determined as $r_1=\frac{a_1}{p_1}$, etc. The weighted average value then appears from the simple formula

$$R=\frac{r_1+2r_2+2r_3+2r_4+r_5}{8} \tag{8}$$

with Q as determined for the discharge and A as found by formula (7) the mean velocity throughout the length of reach tested becomes

$$V=\frac{Q}{A} \tag{9}$$

2. If the areas at the ends of the reach should happen to be nearly alike, the difference in elevation of the water surface Z_1-Z_5 can be taken as the friction loss h_f, and the slope $S=\frac{h_f}{L}$. In this case S would be equivalent to the slope of the water surface. It is believed that all the original experiments of Kutter and many since then have been based on the assumption that S is the slope of the water surface, without determining whether the areas (and hence the velocities) at the ends of the reach were alike. Commonly they are not. This is true for flat slopes and gentle velocities as well as for steep slopes and high velocities, because a difference in velocity heads, at the ends of the reach, of but a few hundredths of a foot may be a large percentage of the fall in the water surface and this difference enters the determination of the friction loss h_f equally with an equivalent fall in the surface.

Assuming the areas and hence the velocities at the ends of the reach are not alike, then, for example, the local velocity $v_1=\frac{Q}{a_1}$ and the velocity head $h_1=\frac{v_1^2}{2g}$. The energy line at location 1 becomes $E_1=Z_1+h_1$, and at location 5 becomes $E_5=Z_5+h_5$. The friction loss $h_f=E_1-E_5$ over the length of reach, L. Therefore, the energy slope

$$S=\frac{h_f}{L}=\frac{E_1-E_5}{L}=\frac{(Z_1+h_1)-(Z_5+h_5)}{L} \tag{10}$$

When the areas at the ends of the reach are reasonably alike there has been no material net change in the investment in velocity head, regardless of minor fluctuations within the reach tested, as evidenced by slightly varying areas in intermediate stations.

From the office platting and computations, R, V, and S have now been determined. These can be substituted in formulas 5 and 6 to solve for n and n'.

NORMAL FLOW AND DEPARTURES THEREFROM

Every conveyance canal is designed with certain assumptions of the four elements S, V, R, and n, looking toward the various combinations of V and A that will multiply together to give the desired design quantity of flow or discharge, Q'. The adopted water prism indicates normal design flow and assumes a uniform flow in a uniform channel between changes in shape or hydraulic elements of the water prism. When the canal is finished, if the assumed value of n and the initial value of n should be exactly alike, the actual normal flow coincides with the design normal. However, this is seldom the case and the water prism at whatever depth in uniform flow, if it actually develops, is in normal flow. In practice, actual flow is seldom uniform over an appreciable reach of canal. Water in a canal is nearly always being either accelerated or retarded within a range of velocities that is sometimes unbelievable. This is evidenced by the fact that nearly all test observations show a difference in the size of the areas of cross section throughout any reach of canal. When velocities are increasing, part of the total energy available at the beginning of the reach has been utilized in the increase in velocity head required at the lower end of the reach over that for the upper end. Conversely, when the velocities are retarded the change in velocity head has been negative and the difference in the necessary velocity heads has been added to the fall of the water surface to make the total fall used in the friction slope.

The deduction from this continual change in average velocity is that the slope of the water surface is not the whole measure of the friction slope and the slope of the energy line must be used. The energy line is located above the water surface by the amount of the velocity head, usually assumed as $1.0\frac{V^2}{2g}$; whereas recent studies of the velocity contours within canal sections indicate that $1.1\frac{V^2}{2g}$ more truly expresses the summation of the velocity heads for the various elements of flow across the water prism. There is no uniformity to the coefficient 1.1, but it is more nearly correct than 1.0 and some computations of carefully explored sections indicate values as high as 1.35, or more.

The term "normal flow" naturally suggests the use of normal velocity, V; normal slope, S; etc., for the various hydraulic elements satisfied when uniform normal flow exists. The question naturally arises, Why is uniform flow not attained as a rule? Such flow would be the rule in the uniform channel regardless of the efficiency of the channel, but this uniformity must extend to uniformity of alinement and of all other attendant conditions. The presence in most canals of tangents and curves in alinement; of various constrictions at checks, drops, bridges, and other structures; of changes in channel materials, such as earth sections changing to concrete-lined sections, or from one type of canal through a flume or siphon pipe and then back to open section again—all tend toward departures from the normal and must be looked for as certainties in canal operation.

DETERMINATION OF ENERGY SLOPE, *S*

In order to bring out the difference between surface slope and energy slope (*52*) in the determination of n, a concrete example is given:

Test No. 260 was made on the Salt River Valley Canal, in Arizona. The various items of the data were as follows:

Length of reach ... feet	900.0		
Discharge, Q, as found by current meter ... second-feet	131.3		
Elements at station 0.			
Assumed elevation of water surface, Z_0 ... feet		90.000	
Area, a_0 ... square feet	38.61		
Velocity $v_0=Q/a_0$ ... feet per second	3.40		
Velocity head, $h_0=v_0^2/2g$		.180	
Elevation of energy line $E_0=Z_0+h_0$ ... feet			90.180
Elements at station 9:			
Fall in water surface between station 0 and 9 ... foot	.681		
Elevation of water surface, $Z_9=90.000-0.681$		89.319	
Area, a_9 ... square feet	43.35		
Velocity $v_9=Q/a_9$ ... feet per second	3.03		
Velocity head, $h_9=v_9^2/2g$ ... foot		.143	
Elevation of energy line $E_9=Z_9+h_9$ ... feet			89.462
Friction loss, between stations 0 and 9, $h_f=E_0-E_9$			.718

Energy gradient, or slope $S=h_f/L=0.718/900=0.000798$.
Surface slope $(Z_0-Z_9)/900=0.681/900=0.000757$.
Constructed, or "design" slope, probably .0008.

The energy slope is more likely to approximate the bed or design slope than is the slope of the water surface. In the example, the value of n computed for the energy slope was 0.0222, while if computed for the slope of the water surface this value was 0.0217. The difference is academic rather than practical for low velocities but for lined canals and flumes, where higher velocities are the rule, it is material and should not be disregarded.

DETERMINING THE PROGRESSIVE VALUES OF n IN THE SAME LOCATION

Sometimes progressive values of maximum carrying capacity of a channel are desirable. These are generally determined in terms of Kutter's n. If the value of n is determined by experiment when the channel is new and clean, then a par value can be computed. Subsequent tests will disclose the progressive relationship to that par value; then, if the capacity is being reduced, various improvement steps can be tried and the net effect of each step determined by observations on one or more test reaches used for all observations. Usually there is a marked difference in the bottom and sides of a channel, especially in the older concrete linings. From periodic tests at various depths of channel, the relative effect of sides and bottom can be definitely determined. Periodic tests, taken over a typical reach and separated only a few days, give an excellent curve of seasonal change in the values of n or in the related maximum carrying capacity. This change in capacity can be pictured on a time-quantity diagram. A second curve

of the seasonal demand or the seasonal supply—whichever controls—will indicate the extent of change necessary in the seasonal capacity. For systems without reservoir storage, seasonal reduction in capacity may not be working hardship, if the available capacity will satisfy the reduced seasonal supply.

The results of periodic tests and the methods used are well exemplified in the improvement of the Los Angeles aqueduct from Owens River, serving Los Angeles with domestic, power and irrigation water. This conduit reaches from Owens Valley to San Fernando Reservoirs, a distance of some 230 miles. It is constructed largely in concrete sections of trapezoidal or rectangular shape, interspersed with many long inverted siphon pipes of concrete and riveted steel and with concrete-lined tunnels. Finished in 1913 the entire aqueduct cost about $25,000,000 (*41, p. 271*); the design hydraulic elements of say the Mojave division canal sections were: Capacity, $Q=431$ second-feet; $V=4.59$, $S=0.00035$, and $n=0.014$. Several years ago it was decided to bring in additional water from Mono Lake by way of Owens Valley, and the city authorities were confronted with the problem of how to do it. The obvious alternatives were to build a second aqueduct, parallel to the first, or to increase the capacity of the first to accommodate the new water. Preliminary observations disclosed that the old aqueduct had deteriorated, in places, so that its capacity had declined to about 400 second-feet. The actual initial capacity—called the par value elsewhere in this bulletin—had not been determined by test. It was several years after completion of the aqueduct that its full capacity was required, and by that time some deterioration had taken place, principally on the bed of the conduit.

Periodic tests, elaborately carried out, resulted in the decision to secure a maximum capacity of 500 second-feet by increasing the capacity of the existing aqueduct. Paradoxically, this was attained by making the channel smaller; that is, an exceptionally smooth lining was laid on the floor of the old concrete (pl. 3, *B*).

Detailed elements of the periodic tests, both before and after improvement of the lining are given in table 1, Nos. 18 to 32. It is to be noted that the increase in capacity was not theoretical, approximately 500 second-feet of water being actually run in the aqueduct. Thus an increase in capacity of about 25 percent was secured at a capital cost of some $320,000, which was only 1.4 percent of the capital cost of the original conduit. This bold undertaking was so completely an appreciation of the value of Kutter's n in the determination of capacity that the methods used in the observations are worth describing.

Proctor [10] discloses the results of the tests leading to the improvement of the aqueduct. In 1927 field parties under his direction regulated a steady flow of 389 second-feet into the aqueduct at Haiwee Reservoir. This was determined by both Venturi meter and current meter observations, which agreed with each other to within 0.56 percent. Similar gagings were made at the various current-meter stations down the aqueduct, and the loss of water in the various reaches was determined. In later computations this loss was prorated between meter stations. The meter measurements were all interpreted through the vertical velocity curves. For the length of the

[10] R. R. PROCTOR, field engineer, Bureau of Water Works and Supply, City of Los Angeles. Unpublished report. The construction was under the direction of J. E. Phillips, engineer in charge of Owens River aqueduct division.

aqueduct, measurements down to the water surface were made from reference points established on manholes or on the edges of holes cut in the concrete cover of the aqueduct. The location of the water surface, was taken as the mean between crest and valley of waves in minute-long observations.

Curves for values of Kutter's n between 0.010 and 0.015 were drafted for each cross section. These were the d, Q curves similar to those shown in figure 1. From these, preliminary figures showing the approximate value of n (assuming uniform flow) and the maximum capacity were determined for each cross section. This procedure also disclosed the bottlenecks where capacity was a minimum and where improvements were to be concentrated. In February 1928, water was

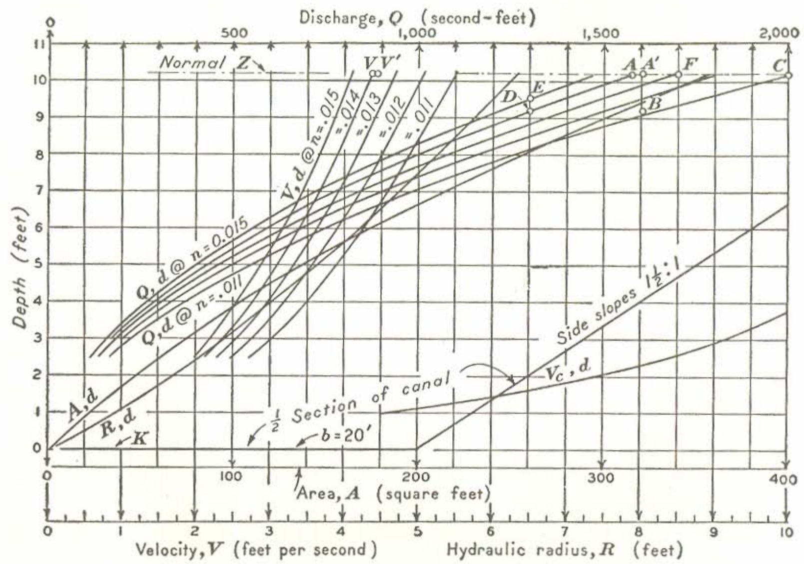


FIGURE 1.—Curves for determination of progressive capacity of a typical lined canal. (Colorado River Aqueduct taken as example.) Basic curves give values of area, A, and hydraulic radius, R, for any depth of water. Secondary curves show values of velocity, V, and discharge, Q, for all reasonable depths and values of n. For comparison, the $V_c d$ curve (which is independent of the value of n) is also given.

turned out of the aqueduct and the inside surface classified in terms of apparent roughness. Cross-sectional measurements were made at each observation point and the energy gradient was computed from point to point. Drop-down curves were encountered in rough reaches followed by smooth ones, and backwater curves in smooth reaches followed by rough ones. Such nonuniform flow was computed for the energy gradients from point to point.

The Freeman division, some 20 miles in length, was found in excellent condition. This division rated the desired extra capacity with but little rehabilitation work. Items Nos. 18 to 24 in table 1 show the values of Kutter's n after this division of the conduit had been in service about 15 years without special treatment of the interior. On the other hand, items Nos. 25 to 32 show similar results for the Mojave division, some 28 miles long, both before repair work and afterward.

In the preliminary study of the aqueduct it was noticeable that the concrete on the bottom had eroded badly in places and in others hardly at all. In fact, much of the difference in capacity, in various reaches, was due to this difference in bottom conditions, and these conditions were traceable to the hardness of the original concrete. Proctor then developed a test procedure. In essentials, a phonograph needle was dulled at the point until about one sixty-fourth of an inch across. This needle was then loaded with 1-, 2-, 3-, 4-, and 5-pound weights. It was found that a satisfactory hardness was achieved when the 4-pound weight was the least that would develop a clean-cut scratch on the concrete, against a straightedge laid on the bottom of the aqueduct (pl. 3, B).

In the rehabilitation work, approximately a 1-to-2 mixture of cement and sand less than one-eighth of an inch in maximum dimension, was used. Over the bottom was placed a mix as dry as would spread uniformly, screeded to a minimum thickness of three-eighths of an inch. A bond with the old concrete was secured by thoroughly brushing a 1-to-1 mixture of cement and sand over the cleaned surface just before the mortar was placed. The mortar was allowed to stand until just before the initial set, excess water being absorbed by the addition of a dry mixture of 1 part sand to 1 of cement. The surface was smoothed, and the drier thoroughly spread with a wooden float. The surface was then finished with a steel trowel after the mortar had started to set, the troweling being delayed as long as possible. It was essential to deposit the mortar at the same rate at which the steel troweling could progress; the trowel work had to be done at the latest possible time so that the operation would not bring water from the mortar to its surface, thus causing a soft wearing surface. This process of finishing by the use of surface drier results in crazing of the surface where the mortar is subject to extreme drying, but none of this took place within the conduit as it remained moist even during periods of shut-down of 2 weeks or more.

EXAMPLE FOR PERIODIC METHOD

For determining the value of n or for the general purpose of observation of progressive capacity of a canal the following outline is given:

Figure 1 shows some of the characteristic curves (57) for the canal section of the new Colorado River aqueduct, conveying water for municipal and irrigation use of the member cities of the Metropolitan Water District of Southern California. This is a concrete-lined canal, 20 feet wide on the bottom with side slopes of 1½ to 1, and design elements as follows: With Manning's $n'=0.014$, then $V=4.45$ for a depth of 10.2 and $S=0.00015$, and $Q'=1{,}605$. With Kutter's formula at $n=0.014$, the value of Q' would be 1,576 second-feet. The diagram is based on Kutter but Manning is quite close, through the range considered. There will be no expansion joints in the lining. Periodic determination of the values of n or of other information concerning this portion of the aqueduct can be quickly made from this set of curves.

Note that the point A' gives design elements by Manning's formula on the curves and A by Kutter's formula. Suppose the value of n shortly after the aqueduct is put in commission should be much lower than the design figure. On a channel of this importance, discharges will be measured as a matter of routine. Suppose the design-Q' of 1,605 second-feet could be run in the canal and found, in long typical reaches, to be but 9.2 feet deep, then the curves (point B) show that the value of $n=0.011+$ has been actually attained. This query then presents itself: If the value of $n=0.011$ holds, what is the maximum capacity of the aqueduct? The point C indicates that nearly 2,000 second-feet could be run in this portion of the aqueduct. If the condition of the aqueduct as a conveyor of water should again be checked, say after 10 years of use, and a flow (discharge) of 1,300 second-feet should be found consistently to occupy a depth of 9.2 feet (point D), then it would appear that the aqueduct had gradually reached a condition equivalent to that assumed in design and that the value of $n=0.014$ holds. If, in say another 10 years, the same flow of 1,300 second-feet occupies a depth of 9.55 (point E) rather consistently, then the aqueduct has depreciated beyond the conditions assumed in design and the value of $n=0.015$ has been reached. Suppose that by this time it is decided a sustained capacity flow of 1,700 second-feet is desirable; then point F shows that the canal surface and hydraulic roughness must be improved until $n=$about 0.013.

Any important canal should be tested to see whether critical depth may be imminent for any condition liable to come to the channel. Critical depth is usually associated with fairly high velocities. For any shape of channel with continuous functions of A, P, R, etc., the velocity head for critical velocity for any depth is given by the formula:

$$h_c=\frac{A}{2T} \tag{11}$$

and

$$V_c=\sqrt{2gh_c} \tag{12}$$

For the metropolitan aqueduct section under consideration the V, d curves for various values of n are given, and it is noted that these curves are quite distant from the curve V_c, d. In other words, values of V_c are so high that no condition of this aqueduct can be imagined that would yield such velocities.

ELEMENTS OF FIELD TESTS TO DETERMINE ROUGHNESS COEFFICIENTS

In table 1 are shown the hydraulic elements of the empirical data, followed by text matter giving brief descriptions of the general conditions at the canals tested. In both table and descriptions the experiments are usually arranged in groups according to the material of the containing channel, while the order within each group follows an ascending value of n. Where several tests were made on the same canal with the same or various discharges of water, tests on that particular canal are not separated.

EXPLANATORY NOTES FOR TABLE 1

Column 1 gives the consecutive numbers, which refer to the order followed in the discussions in the following pages.

Column 2 shows the authority and his experiment number where such was carried. The initials referring to members of the staff of the Bureau of Agricultural Engineering at the time the experiments were made, are as follows: FCS refers to the author, Fred C. Scobey, senior irrigation engineer, in charge of experiments on the flow of water in conduits. At various times he was assisted by E. C. Fortier, P. A. Ewing, F. G. Harden, A. S. Moore, R. H. Wilken, and others. DHB refers to the late Don H. Bark, then in charge of work in Idaho. BPF refers to the late Burton P. Fleming. WBG refers to W. B. Gregory, then head of the department of experimental engineering, Tulane University, La. STH refers to Sidney T. Harding, then irrigation engineer in charge of work in Montana. VMC refers to V. M. Cone, then in charge of work in Colorado.[11] SF refers to the late Samuel Fortier, then Chief of the Division of Irrigation, for citations to experiments he had made some years before (*18*, *19*). FCS+AK refers to experiments made, in informal cooperation, by the author and Arthur Kidder, then engineer for the Pacific Gas & Electric Co.

For experiments by engineers in other agencies, the following symbols are used: BR refers to the United States Bureau of Reclamation. When the reference is followed by initials the engineer reporting the data has been identified: thus D refers to A. L. Darr, F to L. E. Foster, L to E. W. Lane, M to J. S. Moore, and S to W. G. Steward. JBL refers to J. B. Lippincott, consulting engineer, Los Angeles, Calif. (*40*). REB refers to R. E. Ballester, director of irrigation works on the Rio Negro, Argentina (*4*). CCW refers to C. C. Williams, then professor of civil engineering at University of Colorado (*69*). BCC refers to Braden Copper Co., of Chile, reported in correspondence with the writer by A. J. Noerager. See also plate 21, *B*.

JE refers to J. Eppler, of Switzerland (*62*). ES refers to Ettore Scimemi, of Padua, Italy (*53*, *54*), MV refers to Marco Visentini, of Parma, Italy (*68*). RRP refers to R. R. Proctor, field engineer, Department of Water and Power, Los Angeles, Calif.

Column 5 refers to the general shape of the canal cross section, also referred to in figure 2. These data, considered in connection with columns 7 to 9, inclusive, give an idea of the water section.

Column 12 refers to the method of measuring or otherwise determining the discharge at the time of test, Q. In detail: M refers to current meter, I signifies that the integration method was used, VC signifies that the mean velocity was obtained by means of the multiple-point method interpreted through vertical velocity curves, $-2+8$ signifies the mean of the velocities obtained at 0.2 and 0.8 depths in each vertical was accepted as the mean of the vertical, -6 signifies that the velocity obtained at 0.6 of the depth below the surface was accepted as the mean for the vertical. Extensive experiments indicate the discharge computed this way is about 5 percent too high for measurements in artificial channels.

RC signifies the discharge was taken from a rating curve. If the test reach was too far from the gaging station usually an appropriate correction has been made for seepage losses between it and the gaging station. W refers to a weir measurement, under standard conditions. C after the W signifies that a Cipolletti weir was used. Vent. refers to a Venturi meter in a pipe line.

Column 19 shows the various wind conditions; C, signifies calm; U, upstream; D, downstream; and A, across. Where of sufficient importance to affect results seriously, additional information is given in the text.

The other columns are self-explanatory.

[11] About the same time Department Bulletin 194 (*55*) appeared, a similar paper was published by the Colorado Station (*10*). Tests prefixed by VMC are excerpted from that publication.

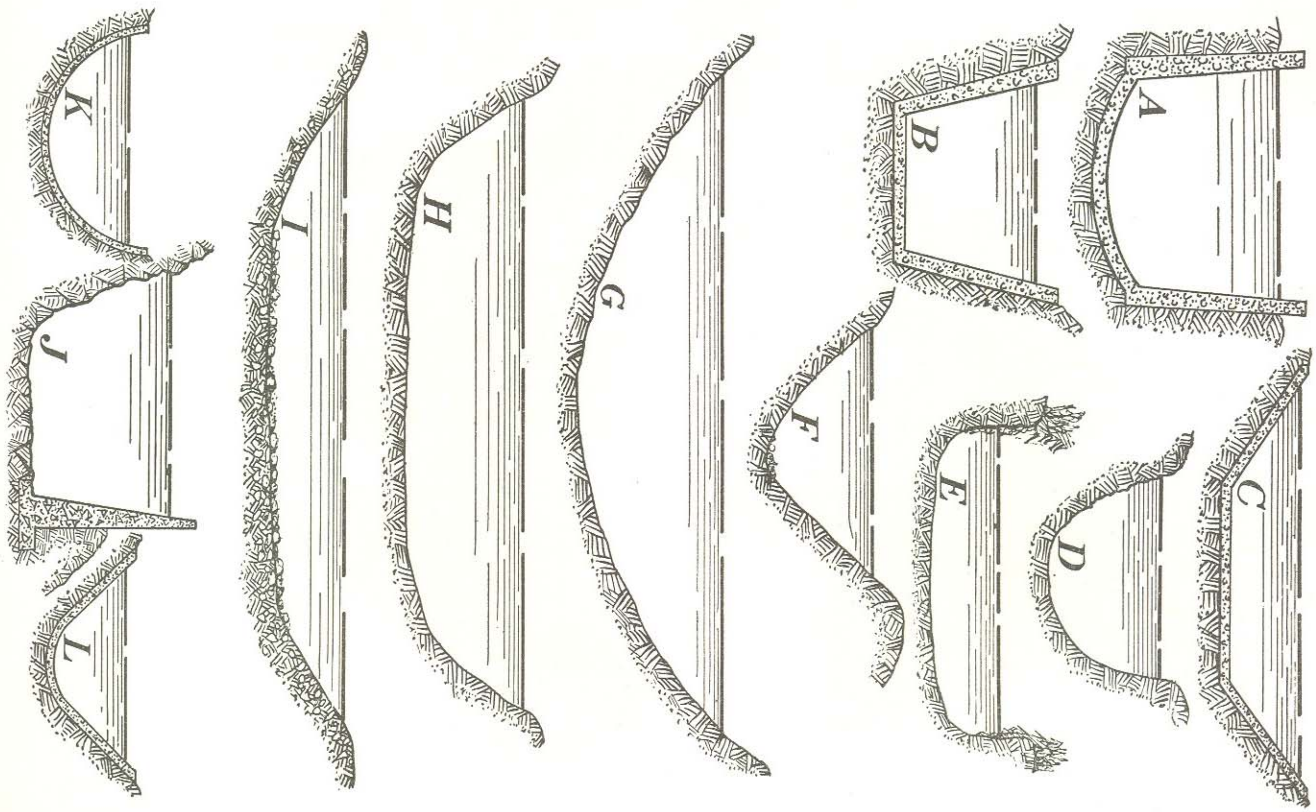


FIGURE 2.—Typical shapes of channels used in classifying data in column 5, table 1.

TABLE 1.—*Elements of experiments determining the coefficient of flow in Chezy's formula and the coefficients of roughness in the Kutter and Manning formulas*

CONCRETE LININGS: POURED OR HAND-LAID CONCRETE

Reference No.	Authority and his experiment No.	Year of test	Name and description of canal	Shape of channel	Length of reach tested, L	Approximate surface width, T	Approximate mean depth	Area of water section, A	Mean velocity per second, V	Discharge per second, Q	Discharge determination method	Hydraulic mean radius, R	Energy slope per 1,000 feet, 1,000 S	Coefficients: Chezy, C	Coefficients: Kutter, n	Coefficients: Manning, n^1	Temperature of water	Wind condition
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
					Feet	*Feet*	*Feet*	*Square feet*	*Feet*	*Cubic feet*		*Feet*	*Feet*				° *F.*	
1	DHB-4	1911	Ridenbaugh canal, Idaho. New, very smooth.	C	1,699.0	22.8	3.10	70.7	4.15	293.6	M-VC	2.81	[1]0.2371	161.5	0.0110	0.0109	----	C
2	FCS-24	1913	Same, tangent, smooth	C	901.0	22.0	2.76	60.6	3.65	221.2	M-VC	2.54	.2508	144.8	.0121	.0121	64	C
3	FCS-24a	1913	Same, tangent and curves. See No. 5	C	1,819.0	22.0	2.76	60.5	3.65	221.2	M-VC	2.54	.2837	136.1	.0129	.0128	64	C
4	BPF-3	1913	Same, tangent and 1 slight curve	C	1,020.6	23.6	3.24	76.5	4.14	316.7	M-VC	2.91	.283	144.5	.0124	----	----	----
5	BR-S-15	1909 to 1911	Same, property of Nampa Meridian irrigation district. Very smooth, troweled. Tests S-19 to S-22 on short selected sections nearly free from gravel. Tests S-15 to 18 and 23-24, some gravel. Test S-25 on reach fairly free from gravel but with several short curves.	C	1,000 to 2,400.[2]	15.5	----	20.6	2.45	50.5	M	1.30	[1].329	118.7	.0132	.0132	----	----
6	BR-S-16			C		20.3	----	44.4	2.32	103.0	M	2.13	[1].1500	129.5	.0130	.0130	----	----
7	BR-S-17			C		23.6	----	68.6	3.35	230.0	M	2.73	[1].1954	145.2	.0122	.0121	----	----
8	BR-S-18			C		26.8	----	95.7	3.99	382.0	M	3.29	[1].2250	146.7	.0124	.0123	----	----
9	BR-S-19			C		15.5	----	20.8	2.43	50.5	M	1.30	[1].3000	123.0	.0127	.0126	----	----
10	BR-S-20			C		19.3	----	42.0	2.45	103.0	M	2.06	[1].1750	128.7	.0131	.0130	----	----
11	BR-S-21			C		22.8	----	68.2	3.37	230.0	M	2.72	[1].1670	158.0	.0112	.0111	----	----
12	BR-S-22			C		26.0	----	93.6	4.08	382.0	M	3.24	[1].2600	140.8	.0128	.0129	----	----
13	BR-S-23			C		25.1	----	87.0	4.32	376.0	M	3.11	[1].2958	142.4	.0126	.0126	----	----
14	BR-S-24			C		23.8	----	76.6	4.15	318.0	M	2.90	[1].3114	138.3	.0129	.0128	----	----
15	BR-S-25			C		15.5	----	22.5	2.60	58.6	M	1.37	[1].2891	130.5	.0122	.0120	----	----
16	JBL-6	1907	Los Angeles conduit, curve	----	700.0	----	----	5.16	2.81	14.5	M-2+8	.83	[1].51	136.7	.0108	.0106	----	----
17	JBL-5	1907	do	----	700.0	----	----	5.0	2.71	13.55	M-2+8	.82	[1].51	132.6	.0111	.0108	----	----
18	RRP	1931	Los Angeles aqueduct, Freeman division. Tests on this division of covered conduit after 18 years service and without special repairs gave certain assurance that other divisions could be improved to meet value of n=0.0125. Side walls showed smooth troweled surface, bottom a good float finish.	A	31,742.0	10.5	4.01	38.1	4.68	178.5	M-VC	2.36	.3988	152.5	.0114	.0116	----	C
19	RRP	1932		A	31,742.0	10.6	5.18	50.7	4.86	246.5	M-VC	2.71	.4000	147.5	.0120	.0119	----	C
20	RRP	1932		A	31,742.0	10.7	6.18	61.9	5.11	316.5	M-VC	2.97	.4000	148.3	.0119	.0120	----	C
21	RRP	1934		A	31,742.0	10.9	7.46	76.2	5.32	405.5	M-VC	3.26	.3985	147.6	.0123	.0123	----	C
22	RRP	1933		A	31,742.0	11.1	8.16	83.5	5.43	453.5	M-VC	3.39	.4000	147.5	.0123	.0124	----	C
23	RRP	1932		A	31,742.0	11.1	8.07	82.7	5.76	476.5	M-VC	3.40	.3992	156.5	.0116	.0117	----	C
24	RRP	1932		A	31,742.0	11.2	8.30	85.4	5.81	496.5	M-VC	3.41	.3988	157.5	.0115	.0116	----	C

[1] Indicates tests where the surface slope was developed and used in the computations.

[2] For Steward tests.

TABLE 1.—*Elements of experiments determining the coefficient of flow in Chezy's formula and the coefficients of roughness in the Kutter and Manning formulas*—Continued

CONCRETE LINING: POURED OR HAND-LAID CONCRETE—Continued

Reference No.	Authority and his experiment No.	Year of test	Name and description of canal	Shape of channel	Length of reach tested, L	Approximate surface width, T	Approximate mean depth	Area of water section, A	Mean velocity per second, V	Discharge per second, Q	Discharge determination method	Hydraulic mean radius, R	Energy slope per 1,000 feet, 1,000 S	Coefficients: Chezy, C	Coefficients: Kutter, n	Coefficients: Manning, n^1	Temperature of water	Wind condition
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
					Feet	*Feet*	*Feet*	*Square feet*	*Feet*	*Cubic feet*		*Feet*	*Feet*				° *F.*	
25	RRP	1927	Los Angeles aqueduct, Mojave division. Tests 25-26-27 before repair work. Tests 28-32 after placing very smooth concrete lining on old bed bringing this division up to the par capacity accepted for Freeman division above. The excellent values of n assured for a long time by making an excessively hard concrete lining.	A	52,796.0	------	7.97	77.95	4.90	382.0	M-VC	3.26	.3593	143.2	.0126	.0127	----	C
26	RRP	1930		A	52,796.0	------	8.25	82.2	4.90	403.0	M-VC	3.30	.3618	141.8	.0128	.0128	----	C
27	RRP	1930		A	52,796.0	------	8.79	86.6	5.00	433.0	M-VC	3.35	.3690	142.3	.0128	.0127	----	C
28	RRP	1932		A	52,796.0	------	5.60	54.0	4.50	243.0	M-VC	2.76	.3548	143.8	.0123	.0123	----	C
29	RRP	1932		A	52,796.0	------	6.58	64.27	4.87	313.0	M-VC	3.00	.3548	149.3	.0118	.0120	----	C
30	RRP	1931		A	52,796.0	------	8.54	86.3	5.48	473.0	M-VC	3.38	.3551	158.2	.0115	.0115	----	C
31	RRP	1932		A	52,796.0	------	8.52	85.2	5.55	473.0	M-VC	3.39	.3551	159.9	.0114	.0114	----	C
32	RRP	1932		A	52,796.0	------	8.80	88.5	5.57	493.0	M-VC	3.39	.3551	160.5	.0113	.0114	----	C
33	FCS-98	1932	Hidalgo and Cameron County, water control and improvement district No. 9	C	2,207.0	6.5	1.30	3.64	1.78	6.49	M-I	.87	.309	108.5	.0133	.0133	83	D
34	BR	1909	Umatilla project, smooth	C	932.0	------	------	2.77	2.06	5.70	M	.58	[1].7	102.0	.013	.0134	----	----
35	BR	1909	Same, on tangent	K	640.0	------	------	28.9	7.10	205.0	M	2.13	[1]1.400	129.0	.0132	.0131	----	----
36	BR	1909	Same, sinuous alinement	K	1,075.0	------	------	28.7	7.15	205.0	M	2.12	[1]1.77	119.0	.0142	.0142	----	----
37	BR	1909	Same, curve	K	220.0	------	------	29.5	6.94	205.0	M	2.15	[1]2.73	90.0	.0189	.0189	----	----
38	BR-D	1922	Klamath project, Oreg. C canal	C	1,180.0	------	------	32.96	3.61	119.0	M-2+8	1.95	.429	124.8	.0135	.0134	----	----
39	BR-D	1922	Same, South Branch (C) canal	C	3,500.0	------	------	34.57	3.85	133.1	M-2+8	2.01	.432	136.0	.0126	.0122	----	----
40	FCS-19	1913	North Side Twin Falls main canal	A	1,900.0	------	------	321.2	8.21	2,637.0	M-VC	5.43	.6389	139.4	.0138	.0143	----	----
41	BR-F	1915	Carlsbad project, N. Mex	C	------	------	2.38	37.0	2.50	92.4	M-VC	1.80	.241	120.0	.0137	.0138	----	----
42	BR-F	1915	do	C	------	------	2.29	30.8	2.96	91.3	M-VC	1.69	.377	117.2	.0139	.0139	----	----
43	FCS-13	1913	Davis and Weber, Utah. Medium smooth.	C	1,000.0	23.0	2.34	53.8	3.94	212.0	M-VC	2.07	.629	109.7	.0154	.0154	64	A
44	BPF-1	1913	do	C	552.0	27.8	2.49	69.29	3.82	264.5	M-VC	2.34	.413	125.7	.014	.0136	----	----
45	BPF-2	1913	do	C	468.5	22.8	1.59	36.19	3.34	120.9	M-VC	1.50	.619	109.0	.0146	.0147	----	----
46	BR-S-1	1909 to 1911	Boise project, Idaho. Main. Straight. Reach roughly troweled. Considerable rock and stone in bed. Tests 46 to 52 inclusive, on section 2. Canal lining designed with n=0.015.	C	1,000.0	58.0	------	302.7	3.34	1,011.0	M	4.89	[1].125	135.0	.0142	.0143	----	----
47	BR-S-2			C	1,000.0	47.0	------	102.6	3.08	316.0	M	2.14	[1].3000	121.4	.0137	.0139	----	----
48	BR-S-3			C	1,000.0	53.7	------	219.4	4.68	1,027.0	M	3.88	[1].3875	120.6	.0154	.0156	----	----
49	BR-S-4			C	1,000.0	48.6	------	133.3	3.57	476.0	M	2.64	[1].3625	115.4	.0152	.0151	----	----
50	BR-S-5			C	1,000.0	46.0	------	92.8	2.64	245.0	M	1.95	[1].2875	111.4	.0149	.0150	----	----
51	BR-S-6			C	1,000.0	44.2	------	64.7	1.84	119.0	M	1.44	[1].2875	90.6	.0170	.0174	----	----
52	BR-S-7			C	1,000.0	55.2	------	246.2	4.91	1,209.0	M	4.22	[1].3125	135.1	.0139	.0140	----	----
53	BR-S-8	1909 to 1911	Same canal, another reach. Much gravel on lined bed for test 53. Very little gravel on bed during tests 54 to 57, inclusive.	C	2,400.0	57.0	------	259.0	3.90	1,011.0	M	4.33	[1].2625	115.6	.0141	.0143	----	----
54	BR-S-9			C	2,400.0	49.1	------	123.3	3.81	470.0	M	2.45	[1].3333	133.4	.0130	.0129	----	----
55	BR-S-10			C	2,400.0	50.4	------	135.0	3.48	470.0	M	2.65	[1].2460	136.3	.0129	.0128	----	----
56	BR-S-11			C	2,400.0	47.0	------	89.1	2.67	238.0	M	1.87	[1].3000	112.7	.0147	.0147	----	----
57	BR-S-12			C	2,400.0	48.4	------	101.3	2.35	238.0	M	2.09	[1].2042	113.8	.0148	.0148	----	----
58	BR-S-13	1909 to 1911	Same canal, another reach, on section 4 with much smoother concrete.	C	2,400.0	53.9	------	205.4	5.00	1,027.0	M	3.64	[1].3154	147.6	.0124	.0126	----	----
59	BR-S-14			C	2,400.0	51.7	------	153.0	2.98	456.0	M	2.89	[1].1470	144.6	.0123	.0125	----	----
60	FCS-94	1931	Water control and improvement district No. 6.	C	1,200.0	5.0	1.9	5.81	1.46	8.50	M-I	.90	.234	101.1	.0141	.0145	----	----
61	DHB-10	1911	King Hill project, Idaho. Rough, curves.	C	925.0	16.5	1.62	21.4	2.55	54.6	M-2+8	1.25	[1].4497	107.5	.0143	.0144	----	----
62	FCS-69	1913	Sanderfer, Calif. Tangent, smooth	B	743.3	3.5	1.43	5.0	3.74	18.7	M-VC	.84	2.375	92.0	.0155	.0155	70	C
63	BR-F-12	1915	Uncompahgre project, Colo	B	2,100.0	14.3	5.66	60.5	8.27	500.0	----------	2.95	1.46	126.2	.0142	.0141	----	----
64	BR-F-13	1915	Same, below 7 drops	B	2,400.0	18.3	4.65	70.2	7.08	497.0	----------	3.02	1.42	108.1	.0165	.0165	----	----
65	BR-F-14	1915	Same, mile post 6	C	500.0	18.5	5.27	71.8	7.14	513.0	----------	3.09	1.18	118.3	.0152	.0152	----	----
66	BR-F-15	1915	Same, below tunnel No. 4	B	425.0	12.83	4.83	50.64	8.63	437.0	----------	2.68	1.74	126.4	.0140	.0139	----	----
67	VMC	1912	Same	B	730.0	14.7	1.71	23.6	4.71	111.3	M	1.41	[1]1.51	102.1	.0155	.0155	----	----
68	JBL-8	1907	Santa Ana, Calif., sand	B	1,000.0	------	------	10.22	2.62	26.79	M-2+8	.82	1.06	89.2	.0157	.0161	----	----
69	BR-M-1	1916	Yakima project, Wash. Mabton Canal. Same. This series of observations was made on the same reach of canal between Apr. 1 (for Mr. Moore's test 1) and Oct. 13, 1916 (for his test 18). The reach contains 6 curves varying in degree from 23° to 75° and having a total length of 441.3 feet. The water surface elevations were derived from hook-gage readings. The gages were installed in vertical concrete wells, cast in the canal bank and connected with the flowing prism by a horizontal pipe. The current meter gaging of discharge was made at the upper end of the reach. Tests 3 to 6 inclusive were made while there were slight obstructions on the canal bed, due to gravel rolling in from the hillside. The increase in n is clearly shown.	C	1,000.0	8.74	1.37	10.06	1.50	15.1	M	1.02	.268	90.9	.0159	.0164	----	----
70	BR-M-2					10.82	2.41	20.3	2.24	45.4	M	1.58	.295	103.7	.0138	.0154	----	----
71	BR-M-3					11.52	2.76	24.2	2.47	59.8	M	1.75	.336	101.9	.0159	.0161	----	----
72	BR-M-4					11.73	2.87	25.40	2.57	65.2	M	1.80	.373	99.1	.0165	.0165	----	----
73	BR-M-5					12.16	3.08	28.00	2.66	74.4	M	1.90	.395	96.9	.0170	.0166	----	----
74	BR-M-6					11.84	2.92	26.05	2.74	71.4	M	1.83	.436	97.1	.0169	.0169	----	----
75	BR-M-7					12.14	3.07	27.8	2.58	71.8	M	1.90	.227	124.3	.0134	.0132	----	----
76	BR-M-8					12.20	3.10	28.2	2.54	71.6	M	1.91	.223	122.9	.0135	.0135	----	----
77	BR-M-9					12.46	3.23	29.79	2.56	76.3	M	1.97	.227	121.1	.0137	.0137	----	----
78	BR-M-10					12.16	3.08	27.97	2.46	68.8	M	1.90	.209	123.6	.0134	.0131	----	----
79	BR-M-11					12.40	3.20	29.3	2.42	71.0	M	1.95	.200	122.2	.0136	.0136	----	----
80	BR-M-12					12.38	3.19	29.3	2.38	69.7	M	1.95	.205	118.8	.0140	.0140	----	----
81	BR-M-13					12.62	3.31	30.83	2.49	76.8	M	2.00	.191	127.2	.0132	.0131	----	----
82	BR-M-14					12.06	3.03	27.3	2.47	67.5	M	1.88	.209	124.7	.0133	.0133	----	----
83	BR-M-15					11.58	2.79	24.59	2.30	56.6	M	1.77	.218	117.2	.0139	.0140	----	----
84	BR-M-16					11.44	2.72	23.78	2.21	52.5	M	1.73	.209	115.9	.0140	.0142	----	----
85	BR-M-17					11.16	2.58	22.2	2.13	47.2	M	1.67	.245	105.2	.0154	.0154	----	----
86	BR-M-18					10.88	2.44	20.5	2.01	41.2	M	1.59	.245	101.5	.0157	.0158	----	----
87	FCS-37a	1913	Orland project, Calif. Lateral No. 12	C	206.0	5.4	.57	3.07	1.86	5.71	M-VC	.54	.99	80.2	.0160	.0167	81	C
88	FCS-37	1913	do	C	755.8	5.4	.62	3.33	1.71	5.71	M-VC	.58	1.184	65.2	.0192	.0208	81	C
89	JBL-7	1907	Colton, Calif. Tangent	---	1,000.0	------	------	6.76	2.27	15.34	M-2+8	.98	[1].70	87.7	.0167	.0170	----	----
90	FCS-11	1913	South Cottonwood Ward, Utah. Sand	B	350.0	2.6	.80	2.07	1.38	2.86	M-I	.52	.694	72.6	.0171	.0185	51	C
91	FCS-55	1913	Modesto main, Calif	B	755.6	21.0	1.52	32.06	3.58	114.8	M-VC	1.38	1.157	89.7	.0174	.0175	75	C
92	FCS-63	1913	Santa Ana main, sandy bed. Deposit	B	1,082.2	11.5	1.41	16.26	1.67	27.16	M-VC	1.22	.321	84.6	.0176	.0182	76	C
		1927	South canal, Pacific Gas & Electric Co. Calif. Series to determine values of n for canal with very sharp bends. Reaches in section A follow each other. Likewise in sections B and C. Reach A1, average curvature, 76 feet radius. Reach A2, radius 450 feet, A3 radius 175 feet, A4 radius 1,100 feet, A5 radius 112 feet, A6 radius 1,100 feet, A7 radius 96 feet, A8 radius 500 feet, B1 radius 78 feet,	C	230.0	------	------	47.05	5.19	244.2	M	2.51	1.100	98.7	.0176	.0176	68	----
93	FCS-AK-A1				230.0	------	------	65.40	5.73	374.6	M	2.98	1.052	102.2	.0174	.0175	68	----
94	FCS-AK-A1				145.0	------	------	45.89	5.32	244.2	M	2.48	1.097	102.0	.0170	.0169	68	----
95	FCS-AK-A2				145.0	------	------	64.5	5.81	374.6	M	2.96	1.062	102.5	.0172	.0172	68	----
96	FCS-AK-A2				178.0	------	------	46.06	5.30	244.2	M	2.48	.910	112.6	.0156	.0155	68	----
97	FCS-AK-A3				178.0	------	------	64.03	5.85	374.6	M	2.96	.978	108.7	.0164	.0164	68	----
98	FCS-AK-A3				174.0	------	------	44.00	5.55	244.2	M	2.42	1.070	109.1	.0159	.0158	68	----
99	FCS-AK-A4				174.0	------	------	62.75	5.97	374.6	M	2.92	1.190	101.1	.0176	.0175	68	----
100	FCS-AK-A4				94.0	------	------	45.14	5.41	244.2	M	2.45	1.298	95.9	.0180	.0180	68	----
101	FCS-AK-A5				94.0	------	------	64.30	5.83	374.6	M	2.96	1.041	105.4	.0169	.0169	68	----

TABLE 1.—*Elements of experiments determining the coefficient of flow in Chezy's formula and the coefficients of roughness in the Kutter and Manning formulas*—Continued

CONCRETE LININGS: POURED OR HAND-LAID CONCRETE—Continued

Reference No.	Authority and his experiment No.	Year of test	Name and description of canal	Shape of channel	Length of reach tested, L	Approximate surface width, T	Approximate mean depth	Area of water section, A	Mean velocity per second, V	Discharge per second, Q	Discharge determination method	Hydraulic mean radius, R	Energy slope per 1,000 feet, $1,000\ S$	Coefficients: Chezy, C	Coefficients: Kutter, n	Coefficients: Manning, n^1	Temperature of water	Wind condition
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
					Feet	*Feet*	*Feet*	*Square feet*	*Feet*	*Cubic feet*		*Feet*	*Feet*				° *F.*	
102	FCS-AK-A5	1927	B2 radius 82 feet, C1 radius 177 feet, C2 radius 500 feet, D radius 86 feet. Sections A and B bed width 6½ feet, with side slopes 1:1 sections C and D, bed width 4 feet 10 inches. Section C had accumulations sand and gravel with few cobbles large as one's head. Also concrete rougher than in sections A and B. For all of the sharp bends on this canal, the high velocities in forward direction were on the inside of curves. Water surface depressed on inside of curves, elevated on outside, but energy line throughout sections A, B, C, and D remarkably straight without local dips opposite bends, showing curvature effect distributed through reach.	C	198.0	------	------	65.38	5.73	374.6	M-------	2.98	.915	109.9	.0162	.0162	68	----
103	FCS-AK-A6				148.0	------	------	46.33	5.27	244.2	M-------	2.50	.912	111.0	.0157	.0156	68	----
104	FCS-AK-A6				282.0	------	------	63.32	5.92	374.6	M-------	2.92	1.241	98.2	.0181	.0177	68	----
105	FCS-AK-A7				332.0	------	------	44.6	5.47	244.2	M-------	2.44	1.289	95.7	.0177	.0177	68	----
106	FCS-AK-A7				196.0	------	------	45.81	5.33	244.2	M-------	2.49	.974	107.7	.0160	.0160	68	----
107	FCS-AK-A8				196.0	------	------	64.0	5.85	374.6	M-------	2.95	1.071	104.0	.0171	.0171	68	----
108	FCS-AK-A8				198.0	------	------	64.72	5.69	368.4	M-------	2.97	1.440	87.5	.0205	.0205	68	----
109	FCS-AK-B1				173.0	------	------	46.24	5.28	244.2	M-------	2.48	1.462	87.7	.0197	.0197	68	----
110	FCS-AK-B1				279.0	------	------	45.46	5.37	244.2	M-------	2.46	1.247	97.0	.0178	.0178	68	----
111	FCS-AK-B2				279.0	------	------	63.72	5.78	368.4	M-------	2.94	1.420	89.4	.0199	.0198	68	----
112	FCS-AK-B2				142.0	------	------	34.46	5.38	185.3	M-------	2.16	1.204	105.5	.0161	.0160	68	----
113	FCS-AK-C1				142.0	------	------	51.42	6.14	315.6	M-------	2.65	1.430	99.7	.0176	.0175	68	----
114	FCS-AK-C1				662.0	------	------	35.40	5.23	185.3	M-------	2.19	1.347	96.4	.0176	.0175	68	----
115	FCS-AK-C2				662.0	------	------	51.50	6.13	315.6	M-------	2.65	1.485	97.8	.0179	.0179	68	----
116	FCS-AK-C2				497.0	------	------	44.11	4.20	185.3	M-------	2.44	.974	86.1	.0200	.0200	68	----
117	FCS-AK-D.				497.0	------	------	52.34	6.03	315.6	M-------	2.67	1.980	82.9	.0211	.0211	68	----
118	FCS-AK-D.																	
119	ES----------	1896	Camuzzoni Canal, Italy----------------					206.6	4.30	888.2	M-------	5.12	.31	108.0	.018	.0182	----	----
120	----do-------	1924	----do---------------------------	----	---------	------	------	485.4	4.82	2,340.0	M-------	8.10	.31	96.0	.022	.0220	----	----
121	FCS-70-----	1913	Los Nietos, Calif. Deposit sand--------	B	600.5	4.4	1.52	6.69	2.89	19.36	M-VC--	.98	1.444	76.8	.0188	.0194	76	A
122	FCS-67-----	1913	Arroyo ditch, Calif. Tangent, moss-----	B	1,000.0	4.1	1.60	6.56	2.83	18.54	M-VC--	.95	1.449	76.2	.0188	.0194	82	C
123	FCS-31a----	1913	North canal, Oreg. Rough, tangent-----	B	240.0	12.9	2.58	33.38	2.94	98.30	M-VC--	1.94	.525	92.5	.0177	.0180	62	D
124	FCS-31b----	1913	Same, tangent and curve----------------	B	1,013.0	12.9	2.58	32.52	3.02	98.30	M-VC--	1.90	.639	86.9	.0187	.0191	62	D
125	FCS-30a----	1913	Same, tangent---------------------------	B	240.0	12.7	2.07	26.26	2.85	74.86	M-VC--	1.63	.629	90.8	.0176	.0178	75	C
126	FCS-30b----	1913	Same, tangent and curve----------------	B	1,013.0	12.7	2.05	26.05	2.87	74.86	M-VC--	1.62	.729	83.5	.0190	.0193	75	C
127	FCS-32a----	1913	Same, tangent---------------------------	B	240.0	12.4	.99	12.25	2.10	25.74	M-VC--	.88	.950	72.5	.0192	.0202	62	C
128	FCS-32b----	1913	Same, tangent and curve----------------	B	1,013.0	12.4	1.02	12.61	2.04	25.74	M-VC--	.91	1.021	67.1	.0206	.0209	62	C
129	JBL-4------	1907	Upper, Riverside, Calif. Two curves----	C	600.0	------	------	6.96	1.22	8.50	M-2+8.-	.70	[1].63	58.0	.0218	.0242	----	----
130	FCS-68-----	1913	Small ditch, cement washed------------	B	575.4	2.7	1.04	2.82	1.07	3.02	M-I-----	.62	.5839	56.1	.0220	.0245	70	C
131	FCS-75-----	1913	Lower, Riverside Water Co., Rough-----	B	699.0	11.0	1.14	12.57	1.88	23.64	M-VC--	1.02	.851	63.8	.0221	.0235	73	C
132	FCS-71-----	1913	Upper, Riverside, Calif. See 129. Sandy.	B	329.6	12.9	2.07	26.72	1.79	47.83	M-I-----	1.60	.482	64.4	.0231	.0250	71	C

SHOT-CONCRETE (GUNITE) LININGS WITH EXCEPTIONS NOTED

133	FCS-91-----	1929	Rossow, near Mission, Tex--------------	L	1,003.8	7.7	2.4	13.32	2.02	27.0	M-I-----	1.39	0.176	129.4	0.0122	0.0122	81	C
134	FCS-97-----	1932	East main, near Edinburgh, Tex---------	L	2,800.0	8.2	2.0	11.74	2.44	28.7	M-I-----	1.23	.382	112.6	.0137	.0137	86	----
135	FCS-90-----	1929	Main, near Progresso, Tex--------------	L	2,483.0	8.3	2.4	13.86	1.93	26.7	M-I-----	1.38	.228	108.4	.0144	.0144	86	----
136	FCS-99-----	1933	Ervine Ranch canal, Calif. Best reach---	K	600.0	------	------	6.72	2.90	19.47	M-I-----	.99	.85	99.7	.0149	.0149	----	----
137	FCS-99a----	1933	Same canal, surface rough and uneven---	K	400.0	------	------	6.08	3.20	19.47	M-I-----	.94	1.428	87.1	.0167	.0169	----	----
138	FCS-99b----	1933	----do---------------------------------	K	900.0	------	------	5.56	3.50	19.47	M-I-----	.97	1.492	84.1	.0173	.0178	----	----
139	FCS-96-----	1932	Main, near Weslaco, Tex----------------	L	1,100.0	7.6	2.4	13.17	1.85	24.4	M-I-----	1.38	2.261	97.6	.0158	.0161	88	----
140	ES----------	1924	Canalĕ di S. Croce, from Piave River----	C	714.4	------	------	49.9	3.61	180.7	M-------	2.17	.53	106.5	.0160	.0160	----	----
141	ES----------	1924	Same, in Italy. 1 curve of 100 meter radius. Built 1920. Sides left as shot. Bed smoothed but covered in places with sand and gravel. Values verify use of $n=0.017$ for untreated shot-concrete.	C	714.4	------	------	64.0	4.13	264.8	M-------	2.59	.58	106.5	.0164	.0164	----	----
142								71.3	4.53	324.4	M-------	2.82	.58	112.1	.0158	.0158	----	----
143								81.8	4.69	384.2	M-------	3.12	.58	110.3	.0162	.0163	----	----
144								87.7	4.92	430.8	M-------	3.18	.60	112.6	.0160	.0164	----	----
145								115.2	5.67	654.6	M-------	3.87	.63	114.8	.0161	.0162	----	----
146	ES----------	1924	Same, farther downstream, much less gradient.	C	1,448.0	------	------	53.3	3.38	180.8	M-------	2.29	.44	106.6	.0160	.0160	----	----
147								71.5	3.71	264.8	M-------	2.83	.39	111.6	.0158	.0158	----	----
148								78.4	4.13	324.4	M-------	3.02	.39	120.3	.0148	.0148	----	----
149								91.0	4.23	384.2	M-------	3.33	.39	117.3	.0155	.0155	----	----
150								97.2	4.43	430.8	M-------	3.44	.39	120.9	.0151	.0151	----	----
151								129.6	5.05	654.6	M-------	4.10	.39	126.2	.0147	.0149	----	----
152	ES-a--------	1924	----do----------------------------------	C	2,257.8	31.99	6.63	125.5	5.90	741.0	M-------	4.07	.380	146.7	.0127	.0129	---	----
153	ES-b--------	1924	Very smooth concrete-------------------		2,257.8	------	9.42	194.6	7.71	1,500.0	M-------	5.25	.418	164.8	.0116	.0116	----	----
154	ES-c--------	1924			2,257.8	------	11.6	254.0	8.27	2,101.0	M-------	6.10	.552	146.7	.0143	.0138	----	----
155	ES-a--------	1924	Same, "cement" lined by hand-----------	C	884.8	48.2	7.48	179.3	4.13	741.0	M-------	4.72	.200	134.0	.0142	.0143	----	----
156	ES-b--------	1924	Reach follows Nos. 152-3-4 above--------		884.8	------	10.2	277.3	5.41	1,500.0	M-------	6.04	.236	144.9	.0131	.0135	----	----
157	ES-c--------	1924			884.8	------	12.4	364.0	5.77	2,101.0	M-------	6.99	.236	146.7	.0140	.0142	----	----
158	ES-a--------	1924	Same, rough, poured concrete, follows Nos. 155-6-7 above. Side slopes 1:1 top to mid-depth, then 2:1 to bottom.	C	1,225.2	54.4	7.97	180.6	4.10	741.0	M-------	4.23	.334	108.7	.0174	.0175	----	----
159	ES-b--------	1924			1,225.2	------	10.66	294.8	5.09	1,500.0	M-------	5.77	.288	129.0	.0150	.0149	----	----
160	ES-c--------	1924			1,225.2	------	12.95	400.0	5.25	2,101.0	M-------	6.59	.335	122.0	.0175	.0172	----	----
161	ES-a--------	1924			2,311.8	------	8.46	200.0	3.71	741.0	M-------	4.53	.274	102.7	.0180	.0177	----	----
162	ES-b--------	1924			2,311.8	------	10.96	309.0	4.86	1,500.0	M-------	5.97	.305	114.8	.0180	.0181	----	----
163	ES-c--------	1924			2,311.8	------	13.16	402.5	5.22	2,101.0	M-------	6.63	.326	112.0	.0190	.0182	----	----
164	ES-a--------	1924	Same (earth cut), follows Nos. 161-162-163 above. Side slopes 2:1. Bed rounded.	C	4,320.6	75.45	8.86	301.2	2.46	741.0	M-------	5.31	.183	77.5	.0250	.0250	----	----
165	ES-b--------	1924					11.12	436.0	3.44	1,500.0	M-------	6.43	.380	69.4	.0290	.0293	----	----
166	ES-c--------	1924					12.86	552.0	3.81	2,101.0	M-------	7.22	.430	68.4	.0290	.0307	----	----
167	FCS-93-----	1931	Lateral N, near McAllen, Tex-----------	P	200.0	3.2	1.4	3.52	3.59	12.6	M-I-----	.74	2.91	77.6	.0176	.0182	88	----
168	FCS-92-----	1931	Same, broomed "gunite"-----------------	P	800.0	5.1	1.90	6.67	1.89	12.6	M-I-----	.99	.374	98.3	.0149	.0149	----	----
169	FCS-89½---	1919	Lower, Lindsay-Strathmore irrigation district.	H	883.9	13.0	2.0	26.09	1.70	44.4	Vent----	1.83	.190	91.4	.0177	.0180	72	----
170	FCS-95-----	1932	West, near Harlingen, Tex--------------	C	1,600.0	9.5	3.5	16.50	1.03	17.0	M-I-----	1.53	.110	79.8	.0187	.0201	85	U

TABLE 1.—*Elements of experiments determining the coefficient of flow in Chezy's formula and the coefficients of roughness in the Kutter and Manning formulas*—Continued

EARTH CANALS

Reference No.	Authority and his experiment No.	Year of test	Name and description of canal	Shape of channel	Length of reach tested, L	Approximate surface width, T	Approximate mean depth	Area of water section, A	Mean velocity per second, V	Discharge per second, Q	Discharge determination method	Hydraulic mean radius, R	Energy slope per 1,000 feet, 1,000 S	Coefficients: Chezy, C	Coefficients: Kutter, n	Coefficients: Manning, n [1]	Temperature of water	Wind condition
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
					Feet	*Feet*	*Feet*	*Square feet*	*Feet*	*Cubic feet*		*Feet*	*Feet*				*° F.*	
171	CCW-14	1908	Interstate, cemented clay	----	----	----	----	207.5	4.75	986.0	M-6	3.98	.17	185.0	.012	.0101	----	----
172	BR	1915	G canal, Minidoka project, Idaho	----	----	----	1.9	21.0	1.24	26.0	----	1.30	.279	65.1	.0226	.0238	----	----
173	BR	1915	Same, section regular, in black loam with	----	----	----	2.4	52.8	1.50	79.2	----	2.02	.208	73.2	.0225	.0227	----	----
174	BR	1915	some weeds on side.				2.7	58.1	1.67	97.1	----	2.17	.208	78.6	.0215	.0214	----	----
175	BR	1915	Same reach. Items for Nos. 172 to 189 inclusive from Bureau Reclamation data card 3.	----	----	----	3.2	71.9	1.81	130.1	----	2.54	.211	78.2	.0222	.0222	----	----
176	BR	1915	Same, section regular, in very slick vol-	----	----	----	2.7	33.8	2.38	80.5	----	1.88	.284	103.0	.0165	.0160	----	----
177	BR	1915	canic ash, no weeds.				2.7	34.6	2.48	85.7	----	1.89	.288	106.4	.0159	.0155	----	----
178	BR	1915	Same reach	----	----	----	2.8	35.1	2.42	85.0	----	1.94	.282	103.5	.0165	.0160	----	----
179	BR	1915	Same project B-1 canal. Sand, some clay	----	----	----	2.7	64.2	1.59	102.0	----	2.02	.182	82.9	.0200	.0201	----	----
180	BR	1915	Same project B canal. Regular, sand	----	----	----	4.4	262.0	2.43	637.0	----	3.63	.200	90.2	.0208	.0204	----	----
181	BR	1915	Same reach, regular, sand	----	----	----	4.6	277.0	2.53	701.0	----	3.81	.197	92.3	.0205	.0206	----	----
182	BR	1915	Same project A-1 canal. Very heavy moss	----	----	----	2.6	44.6	.61	27.2	----	1.69	.230	30.9	.0485	.0523	----	----
183	BR	1915	Same reach, sand and moss	----	----	----	3.2	67.4	1.11	74.9	----	2.37	.250	45.6	.0370	.0369	----	----
184	BR	1915	do	----	----	----	2.7	58.2	1.53	89.1	----	2.07	.190	77.2	.0218	.0216	----	----
185	BR	1915	Same project C canal. Regular, no weeds	----	----	----	4.0	127.5	2.03	259.0	----	3.02	.210	80.6	.0225	.0222	----	----
186	BR	1915	Same project J canal. Not regular	----	----	----	2.4	51.8	1.24	64.2	----	1.84	.195	65.5	.0242	.0250	----	----
187	BR	1915	Same canal, more regular	----	----	----	2.4	49.4	1.30	64.2	----	1.79	.193	70.0	.0226	.0234	----	----
188	BR	1915	Same project Main canal, north side	----	----	----	6.3	508.0	2.03	1,031.0	----	5.15	.152	72.6	.0244	.0269	----	----
189	BR	1915	Same reach	----	----	----	7.0	505.0	2.36	1,192.0	----	5.58	.148	82.1	.0246	.0241	----	----
190	FCS-2	1913	Farmers', Nebraska, cemented clay	G	1,000.0	51.0	2.53	129.1	2.56	330.0	M-VC	2.40	.154	133.0	.0130	.0130	----	C
191	FCS-1	1913	Same, another reach	G	1,200.0	51.0	2.74	139.8	2.22	310.8	M-VC	2.62	.17	105.0	.0164	.0167	----	A
192	SF-15	----	Bear River, Corinne branch	G	100.0	23.9	1.94	46.4	2.36	109.5	M	1.86	[1] .27	105.8	.0155	.0156	----	----
193	VMC	1912	Fort Lyons, silt, very smooth	I	2,600.0	68.0	1.13	75.6	1.86	140.6	M	1.13	[1] .38	89.5	.0165	.0170	----	----
194	FCS-83	1913	Maricopa, Arizona; hard bed	H	900.0	29.0	2.31	66.9	1.28	85.5	M-VC	2.20	.0804	99.3	.0166	.0172	80	A
195	FCS-3	1913	Winter Creek, compact clay	G	800.0	13.5	1.02	13.7	.93	12.8	M-VC	.99	.1341	80.9	.0170	.0184	----	----
196			Evergreen, Salt River project, Arizona.			31.3	----	71.6	2.36	169.0	M-6	2.15	[1] .382	82.3	.0211	.0212	----	----
197			Gaging at middle of reach. First 300			31.4	----	74.5	2.32	173.0	M-6	2.21	[1] .382	79.9	.0210	.0202	----	----
198	BR files	----	feet very irregular; balance very uni-	----	5,000.0	32.2	----	82.0	2.58	212.0	M-6	2.39	[1] .382	85.5	.0200	.0205	----	----
199			form in shape and size. Total curva-			----	----	80.9	2.72	220.0	M-VC	2.38	[1] .372	91.0	.0187	.0188	----	----
200			ture 76° 33′ in flat curves.			----	----	83.8	2.62	220.0	M-VC	2.39	[1] .382	79.0	.0200	.0199	----	----
201			Same reach. Bed of original cemented			----	----	87.0	2.53	220.0	M-VC	2.40	[1] 0.382	82.0	0.0207	0.0211	----	----
202			gravel with sprinkle of clean sand in			----	----	71.0	2.36	167.0	M-VC	2.06	[1] .372	85.0	.0194	.0197	----	----
203	BR files	----	pockets. At sides, roots hold silt. Ex-			----	----	68.5	2.44	167.0	M-VC	2.07	[1] .382	87.0	.0193	.0190	----	----
204			cellent data. Data for Nos. 196-207,	----	5,000.0	----	----	66.2	2.52	167.0	M-VC	2.07	[1] .392	89.0	.0188	.0188	----	----
205			inclusive, listed in Reclamation Record,			30.6	----	60.8	2.32	141.0	M-VC	1.87	[1] .382	87.0	.0190	.0191	----	----
206			July 1913.			30.6	----	62.3	2.23	139.0	M-6	1.92	[1] .382	82.2	.0200	.0209	----	----
207							----	163.2	2.93	478.0	M-6	3.75	[1] .374	78.1	.0238	.0233	----	----
208	CCW-1b	1908	Empire intake, firm gravel and sand	H	400.0	48.0	2.83	135.9	2.94	399.5	M-6	2.84	[1] .25	103.0	.0170	.0173	----	----
209	CCW-1	1908	Same reach, firm gravel and sand	H	400.0	----	----	71.2	2.46	175.5	M-6	1.73	[1] .50	83.6	.0194	.0196	----	D
210	STH-7	1913	Billings Land & Irrigation Co., Mont	G	1,000.0	24.0	2.85	68.4	2.45	167.6	M-VC	2.60	.230	100.0	.0174	.0175	----	----
211	VMC	1912	Jarbeau Power, clay loam	H	900.0	13.6	1.21	16.4	1.96	32.3	M	1.11	[1] .49	83.8	.0176	.0181	----	----
212	STH-5b	1913	Cove, sandy loam, grass	G	600.0	4.5	.53	2.37	1.14	2.70	M-VC	.47	.617	66.6	.0180	.0197	----	----
213	STH-5a	1913	Same canal	G	400.0	5.5	.51	2.78	.97	2.70	M-VC	.50	.460	64.4	.0186	.0206	----	----
214	STH-19	1913	Billings Land & Irrigation Co., silted	H	1,000.0	20.0	2.32	46.5	2.30	106.9	M-VC	2.12	.295	92.0	.0181	.0184	84	A
215	FCS-82	1913	Grand, near Phoenix, Ariz., hard bed	H	1,000.0	29.0	1.99	59.6	2.72	161.8	M-VC	2.04	.438	90.8	.0183	.0185	----	----
216	SF-3	1897	Logan, Hyde Park & Smithfield	H	----	13.7	1.29	17.8	2.58	45.9	M	1.20	[1] .83	81.5	.0184	.0189	----	A
217	STH-18d	1913	Billings Land & Irrigation Co., Mont	H	750.0	25.5	2.47	63.1	1.46	92.1	M-VC	2.34	.111	90.5	.0186	.0190	----	C
218	STH-18b	1913	Same reach, straight with curves beyond			26.5	2.99	79.2	1.88	148.7	M-VC	2.79	.152	91.1	.0193	.0195	----	----
219	STH-18a	1913	ends. Bed covered with fine sand.	H	750.0	27.0	3.03	81.7	2.10	171.6	M-VC	2.80	.1867	91.0	.0194	.0195	----	A
220	STH-18c	1913	Sides, slick clay.			25.0	3.17	82.5	1.67	137.9	M-VC	2.85	.127	88.0	.0199	.0202	60	C
221	STH-6	1913	Same canal. A little fine gravel	----	1,000.0	25.0	2.65	66.2	2.56	169.5	M-VC	2.41	.33	90.8	.0188	.0190	----	C
222	BR-S-37		New York canal, Idaho, good order	----	----	81.8	----	319.0	3.22	1,027.0	M	3.80	[1] .350	88.3	.0211	.0210	----	----
223	BR-S-40		Lizard lateral, Boise project, Idaho	----	----	7.9	----	2.92	.96	2.80	M	.36	[1] .750	58.1	.0190	.0216	----	----
224	BR-S-32a		Same project, N. Nampa lateral. No weeds	----	----	6.7	----	3.66	1.16	4.26	M	.52	[1] .950	52.4	.0225	.0254	----	----
225	BR-S-32b		Same reach, hard pan with gravel, mud	----	----	5.3	----	1.72	1.04	1.79	W-C	.34	[1] 1.870	41.6	.0244	.0290	----	----
226	BR-S-38		Same project, Deer Flat N., moss, weeds	----	----	16.3	----	14.2	.79	11.2	M	.84	[1] .2567	53.9	.0242	.0268	----	----
227	BR-S-33		Same project, S. Nampa lateral, hard pan	----	----	9.2	----	4.03	.87	3.50	W-C	.44	[1] 1.333	36.0	.0293	.0350	----	----
228	BR-S-35	1913	Same reach, bed rough, dragging weeds	----	----	9.4	----	4.77	.44	2.10	M	.56	[1] .180	44.3	.0251	.0298	----	----
229	BR-S-30		Same project, Kennedy lateral, hard pan	----	----	9.2	----	6.03	1.23	7.42	M	.68	[1] 1.00	47.5	.0260	.0292	----	----
230	BR-S-31		Same reach, some gravel, weeds	----	----	5.7	----	2.49	1.25	3.11	M	.42	[1] 1.50	49.8	.0224	.0257	----	----
231	BR-S-39		Same project, Golden Gate canal, sandy	----	----	10.7	----	11.9	1.96	23.3	M	1.06	[1] 1.218	54.4	.0258	.0275	----	----
232	BR-S-36		Same project, Phyllis canal, bed rough	----	----	19.0	----	28.9	1.60	46.2	M	1.60	[1] .440	60.6	.0254	.0266	----	----
233	BR-S-28		Same project, 8-mile lateral, bed rough	----	----	16.2	----	10.6	1.71	18.1	M	.74	[1] 1.70	47.8	.0264	.0292	----	----
234	BR-S-29		Same reach, hard pan, gravel. No grass	----	----	9.0	----	2.48	.97	2.40	M	.30	[1] 1.60	44.6	.0224	.0269	----	----
235	BR-S-35		Same project, Ridenbaugh canal	----	----	20.0	----	49.3	1.18	58.2	M	2.07	[1] .210	56.6	.0286	.0297	----	----
236	FCS-87	1913	Maxwell, Colo. Sandy bed	H	600.0	7.0	.80	5.60	.42	2.36	M-VC	.71	.067	61.3	.0192	.0229	56	C
237	STH-33	1913	Bitter Root Valley Irrigation Co	H	1,000.0	26.0	2.33	60.8	1.85	112.5	M-VC	2.20	.215	85.3	.0196	.0200	----	C
238	VMC	1912	Mesa lateral, Colo. Sedimented	H	550.0	14.8	2.20	27.91	1.51	42.16	M	1.69	[1] .22	78.3	.0200	.0200	----	----
239	FCS-57	1913	Lateral 7, Turlock district, Calif	I	1,000.0	21.0	1.09	22.96	.67	15.4	M-VC	1.04	.098	66.7	.0202	.0225	81	C
240	STH-17	1913	Billings Land & Irrigation Co. Mont	H	400.0	24.0	3.37	80.9	2.56	207.0	M-VC	2.85	.308	86.2	.0204	.0207	----	D
241	CCW-3	1908	Rist & Goss, silted	G	----	----	----	3.60	.91	3.30	M-6	.57	[1] .34	65.0	.0204	.0228	----	----
242	VMC	1912	Willcox. Pebbles and rocks	I	400.0	17.1	.51	8.78	1.74	15.3	M	.33	[1] 3.34	52.1	.0205	.0237	----	----
243	CCW-4	1908	Old Barnes, silt coat	G	----	9.0	.94	8.50	1.17	9.90	M-6	.95	[1] .32	67.0	.0206	.0220	----	----
244	STH-34	1913	Bitter Root Valley Irrigation Co	H	750.0	26.5	----	56.4	2.00	112.5	M-VC	2.03	.312	79.5	.0208	.0211	----	C
·245	SF-6	1897	Logan & Richmond, Utah. Clay	E	100.0	14.9	2.15	32.02	2.14	68.6	M	1.74	[1] .46	75.6	.0211	.0217	----	----

TABLE 1.—*Elements of experiments determining the coefficient of flow in Chezy's formula and the coefficients of roughness in the Kutter and Manning formulas*—Continued

EARTH CANALS—Continued

Reference No.	Authority and his experiment No.	Year of test	Name and description of canal	Shape of channel	Length of reach tested, L	Approximate surface width, T	Approximate mean depth	Area of water section, A	Mean velocity per second, V	Discharge per second, Q	Discharge determination method	Hydraulic mean radius, R	Energy slope per 1,000 feet, 1,000 S	Coefficients: Chezy, C	Coefficients: Kutter, n	Coefficients: Manning, n [1]	Temperature of water	Wind condition
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
					Feet	*Feet*	*Feet*	*Square feet*	*Feet*	*Cubic feet*		*Feet*	*Feet*				° *F.*	
246	STH-35	1913	Bitter Root Valley Irrigation Co	G	1,000.0	27.0	2.21	59.8	1.59	95.3	M-VC	2.08	.20	78.0	.0211	.0216	----	C
247	STH-14	1913	Billings Land & Irrigation Co., Mont	G	400.0	8.4	.98	8.2	.78	6.37	M-VC	.88	.175	62.2	.0212	.0234	----	D
248	WBG-1	1913	Morris, La., bed grassy	I	1,000.0	70.0	2.98	208.5	.43	89.2	M-VC	2.90	.0107	79.6	.0216	.0224	----	A
249	STH-25	1913	Hedge, Mont. Fine gravel, few rocks	H	200.0	16.5	1.94	32.0	2.09	67.0	M-VC	1.80	.44	74.4	.0216	.0221	----	C
250	FCS-78	1913	Birch, Imperial district, Calif. Hard bed.	H	1,000.0	10.0	1.30	13.05	1.34	17.5	M-VC	1.11	.3795	65.0	.0217	.0233	89	C
251	WBG-6	1913	Crowley, La., bed harrowed, grassy	G	1,000.0	27.0	3.08	83.0	.82	68.3	M-VC	2.99	.0345	80.0	.0219	.0224	79	U
252	VMC-b	1912	Bessemer, Colo. Smooth adobe	G	1,600.0	18.2	2.05	37.3	1.55	58.0	M	1.88	[1] .24	72.9	.0219	.0227	----	----
253	VMC-a	1912	Same canal. Smooth adobe	G	1,485.0	16.1	2.31	37.2	1.56	58.0	M	2.04	[1] .36	57.5	.0281	.0293	----	----
254	STH-8	1913	Big ditch, silted	H	1,000.0	10.5	1.17	12.3	1.24	15.2	M-VC	1.06	.357	64.0	.0220	.0235	68	C
255	CCW-2	1908	Louden, Colo. Clean sand	H	----------	25.0	1.49	37.3	1.66	62.0	M-6	1.52	[1] .38	69.0	.0220	.0232	----	----
256	VMC	1912	Mesa lateral (b), Colo. Fine-silt bed	H	600.0	14.7	2.00	27.4	1.47	40.3	M	1.66	[1] .26	70.6	.0220	.0230	----	----
257	FCS-64	1913	Santa Ana main, Calif. Cemented sand	H	1,000.0	16.5	1.15	18.9	1.44	27.2	M-VC	1.06	.481	63.6	.0221	.0237	76	C
258	FCS-80	1913	Central main, Imperial district, Calif	H	998.0	38.0	4.26	161.6	2.08	336.9	M-VC	3.69	.1682	83.8	.0221	.0222	86	C
259	STH-20	1913	Billings Land & Irrigation Co., Mont	H	1,500.0	21.5	2.36	50.9	2.00	102.2	M-VC	2.13	.335	75.0	.0221	.0226	----	D
260	FCS-84	1913	Salt River Valley canal, Ariz	H	900.0	17.0	2.48	42.2	3.12	131.3	M-VC	2.16	.798	75.1	.0222	.0226	79	C
261	CCW-5	1908	Geo. Rist, Colo. Gravel	H	----------	12.0	.92	11.0	1.15	12.7	M-6	.91	[1] .40	66.2	.0224	.0244	----	----
262	STH-2	1913	Big ditch, Mont. Sand bed, mud sides	G	1,000.0	15.5	2.43	40.1	2.09	83.7	M-VC	2.13	.377	73.7	.0225	.0230	----	U
263	STH-38	1913	Bitter Root Valley Irrigation Co	G	400.0	27.0	2.01	56.8	1.63	92.8	M-VC	1.97	.262	71.8	.0226	.0233	62	U
264	FCS-40	1913	Lateral 10, Orland project, Calif	G	568.0	12.0	1.24	14.8	1.78	26.4	M-VC	1.11	.736	62.2	.0228	.0244	----	A
265	REB-1	1925 and 1926	Main, Upper Rio Negro, Argentina. kilometer 8.4.	H	7,126.0	119.1	5.0	529.3	1.87	988.7	M-2+8	4.36	[1] .11	85.1	.0226	.0228	----	----
266	REB-2		Same, kilometer 10.4 in good condition	H	7,222.0	123.0	4.9	512.5	1.93	988.7	M-2+8	4.10	[1] .10	95.3	.0199	.0198	----	----
267	REB-3	1926	Same, kilometer 12.8	H	6,562.0	122.4	4.4	510.6	1.94	988.7	M-2+8	4.10	[1] .074	110.8	.0170	.0170	----	----
268	REB-4	1926	Same, kilometer 14.8	H	6,562.0	117.3	5.2	59.2	1.67	988.7	M-2+8	4.69	[1] .087	88.7	.0243	.0234	----	----
269	REB-5	1925 and 1926	Secondary No. 1. Gravel bed, steep sides	H	656.2	16.4	2.3	33.3	2.78	92.3	M-2+8	1.78	[1] .97	66.6	.0242	.0243	----	----
270	REB-6		Secondary No. 2. Gravel bed, irregular	H	656.2	28.2	3.9	89.8	1.61	144.4	M-2+8	2.89	[1] .16	74.8	.0250	.0242	----	----
271	REB-7		Secondary No. 3. Original shape intact	G	656.2	49.9	4.3	172.8	2.24	387.7	M-2+8	3.31	[1] .16	97.3	.0186	.0190	----	----
272	REB-8	1926	Sandy bed, grassy banks, no silt berms	----	656.2	12.1	2.4	24.7	1.65	40.8	M-2+8	1.68	[1] .21	87.8	.0179	.0185	----	----
273	REB-9	1926	Lateral 1 from secondary No. 6	----	508.5	10.8	2.2	20.0	1.21	24.2	M-2+8	1.42	[1] .30	58.6	.0258	.0269	----	----
274	REB-10	1926	Tests 10 to 14 inclusive on laterials in excellent condition, without silt berms or aquatic growths. However the canal banks have vegetation that drags on the water surface.	----	597.1	16.4	3.2	46.3	1.65	76.6	M-2+8	2.30	[1] .20	81.8	.0218	.0222	----	----
275	REB-11				287.2	13.5	2.5	30.0	1.49	44.6	M-2+8	1.82	[1] .21	76.2	.0217	.0215	----	----
276	REB-12				564.2	11.4	2.1	20.0	1.38	27.7	M-2+8	1.53	[1] .38	57.2	.0264	.0278	----	----
277	REB-13				636.5	15.8	3.0	41.8	1.59	66.6	M-2+8	2.17	[1] .20	76.5	.0217	.0222	----	----
278	REB-14				452.8	13.0	2.4	26.9	1.44	38.7	M-2+8	1.74	[1] .30	62.8	.0249	.0259	----	----
279	REB-15	1926	Tests 15 and 16 on lateral 4 from secondary No. 4. Silt berms, weeds.	----	492.1	16.6	3.1	44.6	1.51	67.6	M-2+8	2.29	[1] .21	68.9	.0243	.0248	----	----
280	REB-16				387.1	16.6	3.3	48.8	1.42	69.1	M-2+8	2.37	[1] .17	70.6	.0238	.0243	----	----
281	REB-17	1926	Silts itself; needs yearly cleaning	----	495.4	10.2	1.6	14.3	1.08	15.5	M-2+8	1.21	[1] .35	52.7	.0271	.0292	----	----
282	REB-18	1926	Rock canal. No desilting required	----	669.3	20.7	3.1	57.8	1.78	103.0	M-2+8	2.41	[1] .33	62.3	.0268	.0273	----	----
283	REB-19	1926	Lateral 4 from secondary 4	----	554.5	9.2	1.9	14.5	1.32	19.1	M-2+8	1.35	[1] .51	50.2	.0291	.0309	----	----
284	REB-20	1926	Same lateral, much vegetation on banks	----	636.5	12.1	2.2	21.6	1.41	30.4	M-2+8	1.58	[1] .48	51.2	.0298	.0313	----	----
285	REB-21	1926	Lateral 1 from secondary 3. Vegetation	----	564.3	13.1	2.4	22.1	.56	12.3	M-2+8	1.53	[1] .21	32.2	.0457	.051	----	----
286	SF-39	1897	Lateral 2 from Bear River, Utah	I	80.0	7.8	.74	5.74	1.38	7.90	W	.71	[1] .875	55.4	.0230	.0254	----	----
287	SF-14	1897	Another lateral from same. Silt and moss.	G	----------	5.5	.71	3.89	1.19	4.63	M	.65	[1] .75	54.0	.0230	.0256	----	----
288	BR files	1897	Indian Bend canal, near Phoenix, Ariz. Bed covered 1 foot clean sharp sand in dunes 0.8 foot high, traveling with flow 2 or 3 feet per hour.	---		37.7	------	68.8	1.76	121.0	M-VC	1.77	[1] .384	67.0	.0230	.0242	----	----
289					5,000.0	38.4	------	82.5	1.88	155.0	M-VC	2.06	[1] .36	66.0	.0249	.0254	----	----
290						------	------	207.0	2.56	530.0	M-6	4.07	[1] .374	65.6	.0314	.0287	----	----
291	FCS-39	1913	South main, Orland project, Calif	H	596.1	21.1	2.12	44.4	1.90	84.3	M-VC	1.93	.3859	69.7	.0231	.0239	82	C
292	FCS-36	1913	Same project. River Branch canal. Few cobbles.	H	593.0	16.5	1.80	29.7	2.82	83.9	M-VC	1.65	1.105	66.0	.0236	.0246	80	C
293	SF-1	1897	Providence, Utah. Medium gravel	H	----------	6.4	.82	5.26	1.90	9.98	M	.71	[1] 1.75	53.8	.0238	.0261	----	----
294	SF-27	1897	Logan, Hyde Park, and Thatcher, Utah	G	100.0	17.6	1.74	30.6	1.97	60.2	M	1.62	[1] .6	63.2	.0246	.0256	----	----
295	WBG-4	1913	Small ditch, La. New	F	350.0	3.5	.69	2.40	.34	.81	M-6	.58	.1014	44.0	.0246	.0309	76	U
296	SF-18	1897	College and City, Utah. Uneven	E	----------	4.4	.80	3.51	1.61	5.66	M	.65	[1] 1.6	50.2	.0247	.0276	----	----
297	FCS-88	1913	Boulder and White Rock, Colo	H	300.0	7.2	.45	3.21	1.0	3.20	M-VC	.43	1.246	43.3	.0248	.0298	55	----
298	ES		See Nos. 164-165-166.															
299	STH-3	1913	Billings Land & Irrigation Co., Mont	H	1,000.0	28.0	3.23	90.3	1.82	164.4	M-VC	2.89	.248	68.2	.0258	.0262	62	D
300	STH-1	1913	Same system	H	1,000.0	27.0	3.45	93.0	2.35	218.3	M-VC	3.02	.38	69.0	.0259	.0261	64	A
301	STH-36	1913	Bitter Root Valley Irrigation Co	G	1,000.0	27.0	2.19	59.1	1.61	95.3	M-VC	2.04	.32	63.0	.0260	.0267	----	C
302	FCS-14	1913	North Ogden, Utah. Gravel	F	800.0	8.5	1.38	11.7	1.72	20.1	M-VC	1.20	.831	54.5	.0262	.0282	58	C
303	FCS-56	1913	Main Branch canal, Turlock district, Calif.	I	1,000.0	28.0	1.26	35.2	.99	35.0	M-VC	1.27	.263	54.4	.0262	.0285	84	C
304	VMC	1912	Rocky Ford. (a) Loose sand bed	H	1,000.0	14.5	1.69	24.4	1.68	41.2	M	1.52	[1] .58	56.5	.0266	.0283	----	----
305	FCS-23	1913	South Side Twin Falls, Idaho, lateral	D	754.0	10.0	1.49	14.9	1.75	26.1	M-VC	1.29	.727	57.1	.0267	.0271	----	A
306	FCS-8	1913	Salt Lake City and Jordan, Utah	E	600.0	9.5	2.19	20.3	1.57	31.9	M-VC	1.57	.4783	57.3	.0267	.0281	62	A
307	FCS-66	1913	Fullerton, Calif. Loose sand bed	G	600.0	12.0	1.15	13.8	1.14	15.7	M-VC	1.03	.493	50.5	.0269	.0297	76	A
308	FCS-86	1913	Farmers', Colo. Few rocks	I	600.0	11.0	.61	6.76	1.41	9.52	M-VC	.59	1.758	43.5	.0270	.0313	59	A
309	STH-12a	1913	Billings Land & Irrigation Co., Mont	G	400.0	8.5	1.16	9.86	1.45	14.3	M-VC	1.02	.812	50.6	.0271	.0296	----	----
310	STH-12c	1913	Same reach. A small lateral, irregular	G	400.0	8.5	.99	7.46	1.23	9.20	RC	.83	.842	46.5	.0277	.0310	----	----
311	STH-12b	1913	Same reach, fringed with grass	G	400.0	7.0	.87	6.12	1.04	6.37	M-VC	.75	.82	42.1	.0294	.0337	----	----
312	FCS-5	1913	Parley's ditch lateral, sandy bed	F	600.0	5.0	.74	3.70	1.20	4.44	M-2+8	.66	1.168	43.2	.0278	.0321	54	U
313	VMC	1912	Bessemer (b), Colo., fine silt, rocks	F	2,002.0	16.4	2.04	33.56	1.26	42.4	M	1.81	[1] .28	55.9	.0280	.0295	----	----
314	FCS-20	1913	South Side Twin Falls, Idaho, lateral	D	718.0	6.5	1.25	8.12	2.50	20.3	M-VC	1.01	2.683	48.1	.0283	.0310	----	C
315	FCS-79	1913	Beech canal, Imperial Valley, Calif	H	1,000.0	8.0	.75	5.97	.87	5.18	M-2+8	.69	.594	42.8	.0290	.0327	89	C
316	FCS-49	1913	Wheeler, Nev., hard bed, loose rock	F	1,164.5	10.5	1.56	16.40	1.18	19.4	M-VC	1.36	.411	49.8	.0292	.0315	66	A
317	FCS-85	1913	Lateral, Arizona canal, near Phoenix	D	700.0	5.5	1.10	6.09	2.36	14.4	M-I	.91	3.076	44.4	.0298	.0339	79	A
318	STH-13c	1913	Billings Land & Irrigation Co., lateral 1	F	600.0	11.0	1.29	14.2	.87	12.4	M-VC	1.15	.29	47.8	.0299	.0319	----	D
319	STH-13a	1913	Same lateral, Mont	G	400.0	7.5	.99	7.42	1.09	8.08	M-VC	.83	.84	41.3	.0308	.0350	----	C
320	FCS-72	1913	Upper, Riverside, Calif., sand, moss	D	294.9	12.0	2.30	27.60	1.73	47.8	M-I	1.88	.634	50.2	.0315	.0330	71	C

TABLE 1.—*Elements of experiments determining the coefficient of flow in Chezy's formula and the coefficients of roughness in the Kutter and Manning formula*—Continued

EARTH CANALS—Continued

Reference No.	Authority and his experiment No.	Year of test	Name and description of canal	Shape of channel	Length of reach tested, L	Approximate surface width, T	Approximate mean depth	Area of water section, A	Mean velocity per second, V	Discharge per second, Q	Discharge determination method	Hydraulic mean radius, R	Energy slope per 1,000 feet, 1,000 S	Coefficients: Chezy, C	Coefficients: Kutter, n	Coefficients: Manning, n^1	Temperature of water	Wind condition
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
					Feet	*Feet*	*Feet*	*Squar feet*	*Feet*	*Cubic feet*		*Feet*	*Feet*				°*F*.	
321	FCS-77a	1913	Lower, Riverside, Calif. Tangent	H	355.0	11.5	1.28	14.78	1.60	23.6	M-VC	1.22	1.065	44.4	.0318	.0347	73	C
322	FCS-77b	1913	Same canal, curve	H	844.4	11.5	1.50	17.2	1.37	23.6	M-VC	1.32	.9095	39.5	.0360	.0395	73	C
323	FCS-59	1913	Modesto main, loose sand	I	930.0	55.0	.86	47.5	.91	43.0	M-VC	.88	.5165	42.7	.0300	.0342	72	C
324	SF-64	1897	Hyrum, Utah, gravel bed	G	----------	5.5	.51	2.82	.88	2.47	W	.49	[1] 1.4	33.6	.0319	.0393	----	----
325	VMC	1912	Bessemer (c), loose stones	F	1,194.0	15.4	2.38	36.6	1.16	42.4	M	2.06	[1].26	50.0	.0321	.0337	----	----
326	FCS-9	1913	Lower, from Big Cottonwood Creek, Utah.	F	1,000.0	8.0	1.23	9.82	1.03	10.1	M-2+8	1.07	.5333	42.8	.0324	.0352	56	C
327	FCS-60	1913	Yosemite Power Co., California	H	662.6	3.5	.56	1.95	1.32	2.57	M	.47	3.679	31.6	.0334	.0415	60	C
328	FCS-62	1913	Golden Rock, Yosemite Power Co	G	537.0	7.2	.81	5.82	1.20	6.97	M-VC	.73	1.629	34.7	.0346	.0407	69	C
329	STH-15	1913	Lateral 1, Billings Land & Irrigation Co	G	400.0	9.5	1.03	9.79	1.12	11.0	M-VC	.96	.93	37.3	.0349	.0397	----	----
330	SF-26	1897	Logan & Benson Ward, Utah, moss	I	----------	21.0	1.39	29.1	.84	24.6	M	1.32	[1].33	40.3	.0352	.0388	----	----
331	FCS-18	1913	Logan & Hyde Park, Utah, gravel	H	550.0	14.5	1.60	23.14	1.18	27.4	M-VC	1.32	.6982	38.9	.0364	.0402	52	C
332	CCW-9	1908	Hillsboro, Colo, rough	H	----------	23.0	2.18	50.1	1.44	72.0	M-6	1.87	[1].60	42.8	.0371	.0387	----	----
333	FCS-53	1913	Lateral 2½, Turlock district, Calif	G	1,020.5	23.0	1.92	44.3	.81	36.0	M-VC	1.80	.217	41.1	.0373	.0401	77	A
334	FCS-25	1913	Perault, Idaho, hard bed, grass	H	610.0	11.5	1.15	13.2	.70	9.28	M-VC	1.06	.398	34.2	.0381	.0440	62	C
335	SF-62	1897	Hyrum lateral, Utah, rocks, moss	F	----------	6.6	.84	5.55	.44	2.44	W	.77	[1].29	29.6	.0393	.0482	----	----
336	FCS-46	1913	Orr, Nev., hard, scoured bed	F	1,426.0	15.0	2.11	31.72	1.44	45.80	M-VC	1.84	0.746	39.1	.0397	.0423	70	A
337	FCS-21	1913	Small ditch in Twin Falls, Idaho	G	308.0	3.9	.44	1.70	.45	.76	W-C	.40	.919	23.3	.0399	.0574	----	----
338	FCS-4	1913	New Rutner, Nebr., gravel bed	H	600.0	5.5	.75	4.15	.65	2.69	M-6	.68	.95	25.6	.0436	.0545	----	C
339	FCS-44	1913	Sullivan and Kelly, Nev., few cobbles	F	1,010.0	13.0	2.17	28.18	1.45	40.9	M-VC	1.80	.894	35.6	.0436	.0463	70	C
340	WBG-2	1913	Roller, La., grass slopes	G	1,000.0	48.0	4.09	196.3	.40	78.35	M-VC	3.90	.023	42.1	.0461	.0446	79	----
341	MV	1933	Canale di Parmigiana-Moglia, Italy	G	5,987.0	52.0	------	548.4	.70	383.8	M	6.89	.0260	52.7	.045	.0400	----	----
342	MV	1933	Same, in mixed clay and sand, mossed	G	5,987.0	52.0	------	633.6	.69	437.2	M	7.48	.0279	47.9	.052	.0437	----	----
343	SF-53	1897	Thatcher lateral, Utah, much vegetation	G	----------	5.0	.51	2.57	.42	1.08	W	.48	[1] 1.070	18.4	.0519	.0715	----	----
344	SF-52	1897	do	F	----------	4.1	.70	2.90	.37	1.08	W	.58	[1].64	19.2	.0529	.0707	----	----
345	WBG-5	1913	Small ditch, Louisiana, grass lined	G	100.0	5.0	.64	3.21	.32	1.01	M	.52	.67	17.3	.0544	.0771	88	----

COBBLE-BOTTOMED CANALS

Reference No.	Authority and his experiment No.	Year of test	Name and description of canal	Shape of channel	Length of reach tested, L	Approximate surface width, T	Approximate mean depth	Area of water section, A	Mean velocity per second, V	Discharge per second, Q	Discharge determination method	Hydraulic mean radius, R	Energy slope per 1,000 feet, 1,000 S	Coefficients: Chezy, C	Coefficients: Kutter, n	Coefficients: Manning, n^1	Temperature of water	Wind condition
346	CCW-13	1908	Beasley, Colo	I	----------	16.0	.46	7.4	1.74	12.9	M-6	.44	[1].93	87.0	.0220	.0149	--------	
347	VMC	1912	Lateral 1 from Rio Grande canal, Colo	I	1,238.0	25.5	1.46	37.22	3.86	143.6	M	1.42	[1] 2.20	68.8	.0221	.0230	--------	
348	VMC	1912	Rio Grande canal, Colo	I	2,000.0	65.4	2.23	146.0	4.84	707.0	M	2.22	[1] 3.07	58.4	.0284	.0289	--------	
349	BCC-2.25		Cachapoal canal, Chile, tests through length of canal at kilometer posts indicated by figures in column 2. Nos. 349-350-351 in cemented gravel and boulders, smooth. No. 352, adobe with some boulders. No. 353 cemented fine gravel, smooth. No. 354, rock cut, cemented fine gravel, smooth.					135.0	3.90	527.0	M	4.38	[1].98	60.7	.0317	.0319	--------	
350	BCC-2.94							107.4	4.91	527.0	M	3.80	[1] 1.02	78.8	.0230	.0235	--------	
351	BCC-4.09							110.9	4.75	527.0	M	3.99	[1] 1.5	61.4	.0307	.0304	--------	
352	BCC-7.03	1928		D	----------	------	------	141.1	3.73	527.0	M	4.43	[1].82	61.9	.0312	.0308	--------	
353	BCC-7.41							93.25	5.65	527.0	M	3.62	[1] 1.5	76.7	.0249	.0240	--------	
354	BCC-8.12							105.3	5.02	527.0	M	3.85	[1] 1.77	60.8	.0308	.0306	--------	
355	BCC-8.31							129.5	4.07	527.0	M	4.38	[1].87	66.0	.0291	.0288	--------	
356	BCC-8.67	1928	Adobe earth, some boulders	D	----------	------	------	120.0	4.39	527.0	M	4.03	[1] 1.23	60.6	.0312	.0318	--------	
357	BCC-9.15	1928	Earth and gravel, stony bed	D	----------	------	------	122.8	4.20	516.0	M	4.04	[1] 1.5	53.9	.0352	.0335	--------	
358	BCC-9.57	1928	do	D	----------	------	------	139.0	3.71	516.0	M	4.19	[1].94	59.1	.0324	.0316	--------	
359	BCC-9.84	1928	do	D	----------	------	------	142.3	3.63	516.0	M	4.20	[1] 1.00	56.0	.0340	.0338	--------	
360	BCC-10.04	1928	do	D	----------	------	------	161.1	3.20	516.0	M	4.74	[1].70	55.5	.0356	.0370	--------	
361	BCC-10.23	1928	do	D	----------	------	------	121.3	4.25	516.0	M	4.21	[1] 1.7	50.2	.0382	.0421	--------	
362	BCC-10.37	1928	Cemented gravel and boulders, smooth	D	----------	------	------	118.6	4.35	516.0	M	4.06	[1] 1.30	59.9	.0311	.0313	--------	
363	BCC-10.60	1928	Earth and boulders, rough stony bed	D	----------	------	------	127.3	4.05	516.0	M	4.33	[1] 1.80	45.9	.0423	.0414	--------	
364	BCC-10.94	1928	Cemented gravel and boulders, smooth	D	----------	------	------	128.7	4.01	516.0	M	4.36	[1] 1.00	60.7	.0326	.0312	--------	
365	BCC-10.99	1928	Earth and boulders, rough stony bed	D	----------	------	------	129.1	4.00	516.0	M	4.32	[1] 1.54	49.0	.0395	.0405	--------	
366	BCC-11.28	1928	Cemented gravel and boulders, smooth	D	----------	------	------	110.8	4.66	516.0	M	4.01	[1] 1.30	64.5	.0288	.0290	--------	
367	BCC-11.41	1928	do	D	----------	------	------	106.5	4.84	516.0	M	3.86	[1] 1.40	65.9	.0270	.0282	--------	
368	BCC-11.50	1928	do	D	----------	------	------	93.7	5.54	516.0	M	3.61	[1] 1.93	66.3	.0278	.0277	--------	
369	BCC-11.73	1928	Cemented fine gravel, smooth	D	----------	------	------	103.7	4.98	516.0	M	3.84	[1] 1.10	76.6	.0243	.0243	--------	
370	BCC-11.85	1928	do	D	----------	------	------	104.3	4.95	516.0	M	3.91	[1] 1.10	75.5	.0258	.0247	--------	
371	MV	1892	Cavour, Italy. Finished 1866. Tests by Bazin. Rocky bed. Grassed sides.	H	----------	71.5	------	448.3	3.10	1,390.0	----------	5.17	[1].290	80.1	.0247	.0246	--------	
372	MV			H	----------	76.8	------	718.6	3.70	2,659.0	----------	7.31	[1].290	80.4	.0262	.0259	--------	
373	MV-1	1933	Same, tests 41 years later by Visentini	H	16,400.0	65.0	------	794.0	4.22	3,347.0	M	8.07	.258	92.3	.023	.0229	--------	
374	MV-2	1933	Same, bed covered, small cobbles	---	16,400.0	------	------	825.0	4.40	3,632.0	M	8.31	.310	86.7	.025	.0246	--------	
375	SF-63	1897	Hyrum, Utah	H	----------	5.0	.37	1.87	.84	1.57	W	.35	[1] 1.3	38.9	.0260	.0320	--------	
376	STH-31	1913	Bitter Root Valley Irrigation Co., Mont	H	600.0	27.0	2.32	62.57	2.27	142.0	M-VC	2.27	.55	60.6	.0262	.0283	--------	
377	STH-4	1913	Billings Land & Irrigation Co., Mont	H	1,000.0	24.0	2.67	64.10	2.67	171.0	M-VC	2.38	.72	64.2	.0264	.0269	62	D
378	CCW-7	1908	Loveland and Greeley, Colo	I	----------	34.0	1.74	59.20	1.69	100.0	M-6	1.69	[1].50	58.2	.0267	.0280	--------	
379	FCS-10	1913	Upper, from Big Cottonwood Creek, Utah.	H	950.0	11.0	1.39	15.28	1.77	26.9	M-VC	1.17	1.012	51.2	.0277	.0299	51	C
380	FCS-15	1913	Logan and Northern, Utah, grassy banks	D	630.0	16.2	2.60	42.72	1.77	75.7	M-VC	2.25	.37	61.3	.0270	.0272	51	C
381	VMC	1912	Rio Grande lateral No. 1, Colo	I	5,280.0	43.7	1.87	81.6	4.66	380.0	M	1.87	3.66	56.3	.0284	.0294	--------	
382	FCS-51	1913	Reno, of Reno Light & Power Co., Nev., riprap.	D	800.0	18.0	3.92	70.59	3.57	252.0	M-VC	3.02	1.129	61.1	.0291	.0294	----	D
383	CCW-12	1908	Beasley, Colo., gravel bed, log sides	I	----------	24.0	.50	12.10	1.10	13.3	M-6	.48	[1] 1.4	42.5	.0320	.0320	--------	
384	FCS-43	1913	Sullivan and Kelly, Nev., laid wall	J	670.0	10.5	2.45	25.7	1.59	40.9	M-VC	1.81	.603	48.2	.0324	.0342	70	A
385	SF-55	1897	Smithfield lateral, Utah	H	----------	5.1	.58	2.97	.43	1.29	W	.52	[1].353	32.0	.0329	.0417	--------	
386	SF-47	1897	Logan and Hyde Park, Utah	E	----------	1.8	.35	.63	1.35	.85	W	.27	[1] 9.91	25.9	.0337	.0461	--------	
387	SF-33	1897	Hyrum lateral, Utah	I	----------	3.6	.22	.80	1.02	.81	W	.20	[1] 12.12	20.7	.0365	.0548	--------	
388	SF-57	1897	Smithfield lateral, Utah	I	----------	4.6	.24	1.11	1.33	1.48	W	.23	[1] 17.1	21.1	.0377	.0550	--------	
389	FCS-42	1913	Cochrane, Nev	G	733.8	14.0	1.58	23.06	1.18	27.2	M-VC	1.39	.695	37.9	.0379	.0416	71	U
390	FCS-89	1913	Beasley, Colo., very little grass	I	889.2	14.0	.71	9.97	1.82	18.1	M-VC	.65	5.84	29.6	.0383	.0468	52	A
391	FCS-52	1913	Capurro, Nev	E	300.0	3.2	.86	2.74	.35	.96	W-C	.56	.3367	25.6	.0403	.0527	----	C

TABLE 1.—*Elements of experiments determining the coefficient of flow in Chevy's formula and the coefficients of roughness in the Kutter and Manning formulas*—Continued

COBBLE-BOTTOMED CANALS—Continued

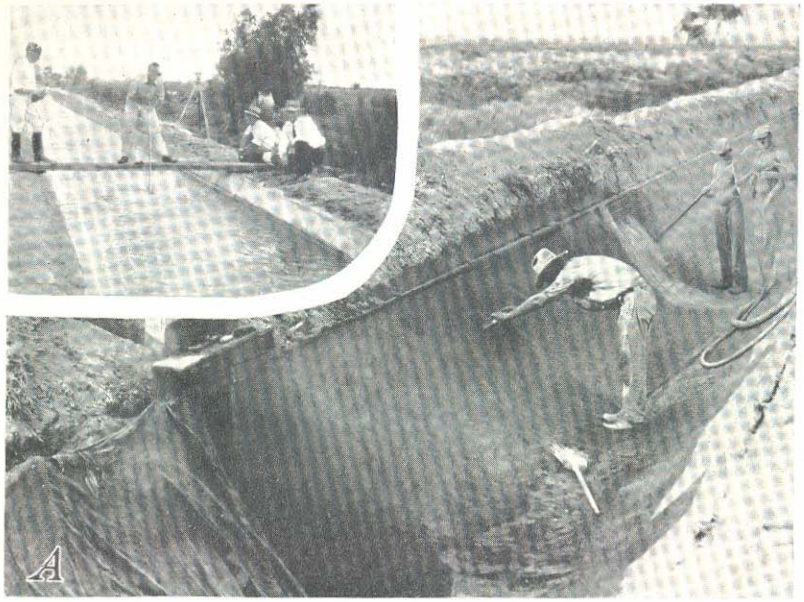
Reference No.	Authority and his experiment No.	Year of test	Name and description of canal	Shape of channel	Length of reach tested, L	Approximate surface width, T	Approximate mean depth	Area of water section, A	Mean velocity per second, V	Discharge per second, Q	Discharge determination method	Hydraulic mean radius, R	Energy slope per 1,000 feet, 1,000 S	Coefficients: Chezy, C	Kutter, n	Manning, n [1]	Temperature of water	Wind condition
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
					Feet	*Feet*	*Feet*	*Square feet*	*Feet*	*Cubic feet*		*Feet*	*Feet*				°*F.*	
392	SF-24	1897	Brigham City Electric Light Co., Utah	F	----	10.7	1.91	20.44	1.52	31.1	----	1.62	[1]1.15	35.2	.0424	0.0460	----	----
393	SF-56	1897	Smithfield lateral, Utah, very uneven	I	----	4.6	.23	1.06	1.10	1.17	----	.23	[1]17.0	17.9	.0423	.0648	----	----
394	SF-46	1897	Brigham City, Utah, much vegetation	G	----	19.6	1.82	35.76	.65	23.4	M	1.74	[1].28	29.6	.0499	.0553	----	----
			SIDEHILL CUTS WITH RETAINING WALLS															
395	STH-23	1913	Hedge canal, Mont., plastered wall	J	250.0	14.0	3.08	43.10	1.58	68.1	M-VC	2.31	.132	90.5	.0185	0.0190	55	C
396	STH-27	1913	Same, earth and gravel bed	J	450.0	13.0	3.76	35.9	1.85	66.5	M-VC	2.10	.30	73.8	.0225	.0229	----	----
397	STH-9	1913	Cove canal, Mont., concrete floor	J	900.0	12.0	1.68	20.21	2.30	46.6	M-VC	1.45	.80	67.5	.0228	.0229	----	----
398	FCS-17a	1913	Logan, Hyde Park, and Smithfield, Utah	J	337.0	13.0	1.73	22.49	3.12	70.2	M-VC	1.48	1.852	59.7	.0256	.0267	----	----
399	FCS-17b	1913	Same, another reach	E	100.0	12.5	1.78	22.22	3.16	70.2	M-VC	1.44	2.37	54.2	.0278	.0293	55	C
400	STH-26	1913	Hedge canal, Mont., concrete wall, floor	J	300.0	12.5	2.88	36.0	1.85	66.5	M-VC	2.13	.44	60.5	.0269	.0280	----	----
401	MV	1931	Fossa di Pozzolo, Italy, bed and sides rocky, masonry wall lower side.	J	3,918.3	42.65	3.81	128.7	3.50	380.6	M	3.47	1.067	53.4	0.033	.0238	----	----
402	MV				3,918.3	42.65	4.30	164.0	3.47	569.4	M	3.50	1.087	57.8	.032	.0328	----	----
403	MV	1931	Same reach. See plate 12-A	J	3,918.3	42.65	4.40	197.1	4.04	793.1	M	4.01	1.112	62.7	.030	.0312	----	----
			MISCELLANEOUS SECTIONS															
404	FCS-74	1913	Lower, Riverside Water Co., Calif	E	200.0	10.5	1.38	14.45	1.32	19.0	M-VC	1.17	.45	57.4	0.0249	0.0267	73	C
405	FCS-73	1913	Same, sandy bed, one side planked		750.0	10.0	1.52	15.15	1.26	19.0	M-VC	1.24	.5253	49.1	.0291	.0315	73	C
406	BR-S-26		Rossi Mill, Idaho, sides almost vertical of rough boards, grass in cracks.	J	520.0	12.25	------	41.2	3.64	150.0	M	2.53	[1].7115	85.8	.0202	.0202	----	----
407	BR-S-27				520.0	12.66	------	49.3	4.18	206.0	M	2.65	[1].8654	87.3	.0200	.0200	----	----
408	FCS-17c	1913	Logan, Hyde Park, and Smithfield, Utah	E	265.0	9.5	2.15	20.40	3.44	70.2	M-VC	1.52	3.0	50.9	.0298	.0314	55	C
409	JE	1905	Power canal, city of Aarau, Switzerland	C	1,066.0	52.1	------	405.0	3.32	1,346.0	M-VC	6.62	.120	118.0	.0173	.0173	----	----
410	ES-I to II	1924	Power canal, from Cellina Torrente, Italy, built 1905, A rock cut into steep mountain side, with rough concrete retaining wall. Curves are frequent. 4 reaches tested, following each other. Some moss in canal with irregular bed. Energy gradient used for slope. Nonuniform flow, becoming deeper at lower ends of reaches.	J	2,613.5	13.25	5.05	66.25	2.82	186.8	M	2.86	.314	91.1	.0190	.0189	----	----
411	ES-I to II				2,613.5	------	6.40	84.60	3.64	308.2	M	3.24	.452	99.7	.0193	.0191	----	----
412	ES-I to II				2,613.5	------	7.55	99.87	3.97	396.5	M	3.30	.464	98.0	.0179	.0185	----	----
413	ES-I to II				2,613.5	------	8.79	116.6	4.27	499.3	M	3.78	.565	95.8	.0201	.0201	----	----
414	ES-II to IV				1,587.2	------	5.71	75.7	2.46	186.8	M	3.08	.289	82.8	.0216	.0222	----	----
415	ES-II to IV				1,587.2	------	6.69	88.58	3.48	308.2	M	3.32	.496	82.0	.0212	.0212	----	----
416	ES-II to IV				1,587.2	------	7.74	103.1	3.87	396.5	M	3.56	.599	83.4	.0220	.0220	----	----
417	ES-II to IV				1,587.2	------	8.86	117.4	4.27	499.3	M	3.79	.620	87.8	.0211	.0211	----	----
418	ES-IV to IX				2,357.5	------	6.36	84.9	2.23	186.8	M	3.21	.195	89.2	.0202	.0203	----	----
419	ES-IV to IX				2,357.5	------	6.86	91.1	3.38	308.2	M	3.37	.459	85.2	.0220	.0213	----	----
420	ES-IV to IX				2,357.5	------	7.74	102.8	3.87	396.5	M	3.57	.593	87.7	.0210	.0210	----	----
421	ES-IV to IX				2,357.5	------	8.79	116.6	4.30	499.3	M	3.78	.584	91.1	.0203	.0204	----	----
422	ES-IX to X				1,167.3	------	7.02	93.3	2.00	186.8	M	3.37	.169	83.9	.0225	.0218	----	----
423	ES-IX to X				1,167.3	------	7.09	94.1	3.28	308.2	M	3.42	.422	86.4	.0211	.0218	----	----
424	ES-IX to X				1,167.3	------	7.76	103.0	3.84	396.5	M	3.57	.5902	83.4	.0220	.0221	----	----
425	ES-IX to X				1,167.3	------	8.63	114.7	4.36	499.3	M	3.74	.787	79.7	.0230	.0231	----	----
426	FCS-AK	1929	Drum canal, Pacific Gas & Electric Co., Calif.	D	600.0	------	------	102.6	4.82	494.5	M	3.49	1.366	69.8	.0262	.0260	----	----
427	BCC-8.12	1928	Cachapoal canal, Chile. See 354, et seq	D	------	------	------	105.0	5.02	527.0	M	3.85	[1]1.77	60.8	.0308	.0306	----	----
			MASONRY-LINED															
428	FCS-100	1926	Main, Deschutes municipal district, Oreg.	B	700.0	12.2	------	21.60	3.27	70.7	M-I	1.46	1.297	75.2	0.0207	0.0210	57	C
429	FCS-98	1923	Yakima Valley, Wash	J	1,100.0	5.69	2.49	14.18	4.70	66.6	M-I	1.42	1.486	102.5	.0154	.0154	67	C
430	BR	------	Cottonwood, rubble sides	----	------	5.7	------	4.76	6.17	29.4	------	.86	[1]5.575	89.3	.0163	.0163	----	----
431	FCS-29	1913	Jacobs Ditch, Idaho, rubble sides	E	225.0	4.2	1.40	5.90	3.29	19.4	M-VC	.86	1.367	96.3	.0149	.0151	63	C
432	FCS-27	1913	Same ditch, unchinked sides	E	280.0	7.4	1.77	13.06	1.49	19.4	M-VC	1.23	.471	61.9	.0235	.0249	63	C
433	FCS-28	1913	Same ditch, plastered sides	E	280.5	5.1	1.70	8.62	2.26	19.4	M-VC	1.05	1.597	55.5	.0250	.0271	63	C
434	FCS-45	1913	Orr ditch, Nev., mortar-laid masonry	E	213.6	11.5	2.26	26.02	1.76	45.80	M-VC	1.78	.636	52.4	.0298	.0314	----	C
			CONCRETE CHUTES															
435	BR	1912	Sulphur Creek Wasteway, Wash. Series of tests on 2 reaches of this steep chute. Some doubt as to year of tests, perhaps in 1911. Nos. 435 to 442 inclusive on reach with 2° curve (radius 2,865 feet). Concrete cast in wood forms and not retouched. Constructed slope, this reach, 0.0206.	C	900.0	------	------	3.98	11.0	43.8	------	0.67	[1]20.5	94.3	0.0147	0.0148	----	----
436								4.23	12.4	52.5	------	.69	[1]20.5	104.0	.0136	.0134	----	----
437								6.90	10.7	73.7	------	.94	[1]20.8	76.4	.0188	.0192	----	----
438								10.38	12.8	133.0	------	1.21	[1]21.3	79.6	.0190	.0193	----	----
439								12.83	14.8	190.0	------	1.38	[1]21.8	85.4	.0183	.0184	----	----
440								14.81	15.9	235.0	------	1.50	[1]21.7	88.1	.0180	.0179	----	----
441								12.67	19.1	242.0	------	1.36	[1]20.6	114.0	.0140	.0138	----	----
442								12.79	19.3	247.0	------	1.37	------	114.0	.0140	------	----	----
443	BR	1912	Same chute, on tangent. On Yakima project. Section, semicircular on radius of 4 feet. This reach follows one above. Constructed slope, 0.0145. This chute designed with n=0.013. Further data, Citation No. 15, p. 264 in vol. 3. See plate 8, B.	C	1,300.0	------	------	3.19	13.7	43.8	------	.58	[1]14.6	149.0	.0097	.0092	----	----
444								6.34	11.6	73.7	------	.90	[1]14.7	101.0	.0145	.0141	----	----
445								4.01	13.1	52.5	------	.67	[1]14.4	133.0	.0109	.0105	----	----
446								4.64	16.9	78.5	------	.73	[1]14.5	163.0	.0093	.0087	----	----
447								9.58	13.9	133.0	------	1.15	[1]14.9	106.0	.0146	.0144	----	----
448								11.51	16.5	190.0	------	1.29	[1]14.6	120.0	.0132	.0129	----	----
449								13.30	17.7	235.0	------	1.41	[1]15.2	121.0	.0133	.0131	----	----
450								10.11	20.4	247.0	------	1.33	[1]14.4	148.0	.0110	.0105	----	----

TABLE 1.—*Elements of experiments determining the coefficient of flow in Chezy's formula and the coefficients of roughness in the Kutter and Manning formulas*—Continued

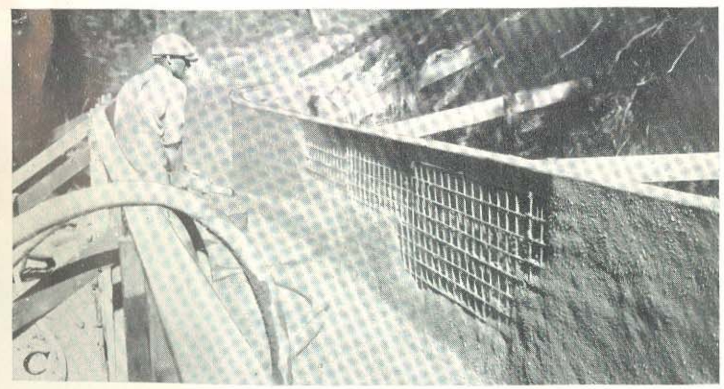
CONCRETE CHUTES—Continued

Reference No.	Authority and his experiment No.	Year of test	Name and description of canal	Shape of channel	Length of reach tested, L	Approximate surface width, T	Approximate mean depth	Area of water section, A	Mean velocity per second, V	Discharge per second, Q	Discharge determination method	Hydraulic mean radius, R	Energy slope per 1,000 feet, 1,000 S	Coefficients: Chezy, C	Coefficients: Kutter, n	Coefficients: Manning, n^1	Temperature of water	Wind condition
1	2	3	4	5	6	7	8	9	10	11	12	14	15	16	17	17	18	19
					Feet	*Feet*	*Feet*	*Squar) feet*	*Feet*	*Cubic jeet*		*Feet*	*Feet*					
451	VMC	1912	South canal Uncompahgre project, Colo.	B	201.0	10.4	0.56	5.78	15.5	89.8	M	.51	[1] 71.8	80.8	.0158	.0164	----	----
452	VMC	1912	Same, 7 miles downstream from 451	B	142.0	13.3	1.40	5.28	11.3	59.7	M	.38	[1] 72.30	68.4	.0171	.0184	----	----
453	BR-F-11	1915	Same, bed pitted, patches plaster gone	B	375.0	10.99	2.99	28.3	20.2	573.0	RC	1.93	21.14	100.0	.0168	.0166	----	----
454	BR-L	1931	Same. All reaches begin at station 109+00.	B	350.0	------	------	5.11	29.3	150.0	RC	.60	129.7	105.0	.0131	.0129	----	----
455	BR-L	1931	Same. Value of n computed from average values of R and V. All flows faster than critical velocity (Vc).	B	460.0	------	------	7.30	27.4	200.0	RC	.91	101.6	90.0	.0162	.0163	----	----
456						------	------	8.83	28.3	250.0	RC	1.05	105.5	85.0	.0175	.0176	----	----
457	BR-L; BR-L; BR-L	1931	Same. Usual flows from 350 to 400 second-feet. Same, for all test reaches, bed and lower sides eroded and rough. Discharge. Q by recording gage 1¾ miles above chute. 1 percent deducted for loss. Cross sections taken with water out of canal. This canal begins at outlet of Gunnison tunnel.	B	600.0	------	------	11.3	26.6	300.0	RC	1.20	78.0	86.8	.0176	.0177	----	----
458					600.0	------	------	12.4	28.2	350.0	RC	1.28	78.1	89.0	.0174	.0174	----	----
459					600.0	------	------	13.6	29.4	400.0	RC	1.35	78.3	90.6	.0173	.0173	----	----
460					700.0	------	------	15.67	28.7	450.0	RC	1.37	66.9	94.7	.0166	.0166	----	----
461					700.0	------	------	16.84	29.7	500.0	RC	1.43	67.0	96.2	.0165	.0164	----	----
462					700.0	------	------	17.74	31.0	550.0	RC	1.47	67.1	98.8	.0162	.0160	----	----
463					700.0	------	------	18.58	32.3	600.0	RC	1.51	67.2	101.0	.0159	.0157	----	----
464					700.0	------	------	19.46	33.4	650.0	RC	1.54	67.3	104.0	.0155	.0153	----	----

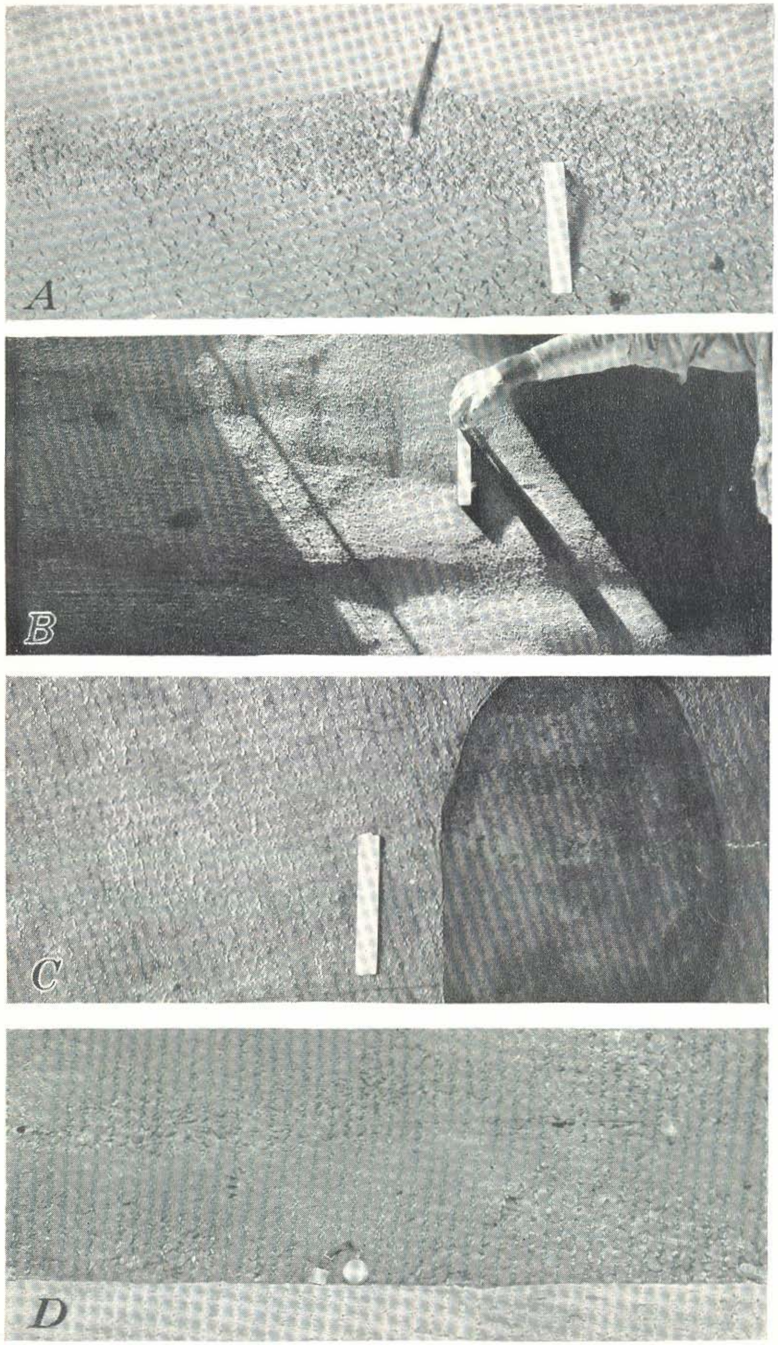
A, Main canal, Kittitas division, Yakima project, Washington. Constructed about 1929. Excellent example of concrete poured behind screeding and smoothing platform. Designed with $n=0.014$; the initial value was about 0.013 of less. Some 615 second-feet of water was in canal when photographed. *B*, A lateral in California, lined more than 20 years before the picture was taken. Low velocity of warm, clear water conducive to aquatic growths. Note weeds growing in expansion joint cracks. Bureau of Reclamation photographs.



A, Canal being lined with concrete shot from a gun. Note rough initial surface being improved by striking with a rectangular blade. If the surface is untouched the value of n is about 0.017, while if surface is treated as shown, n is reduced to less than 0.013 in many cases. Inset shows a test on this type of canal. (No. 133.) The debris resulting as rebound and from the striking process must be carefully removed, as it has little cementing value. *B*, A more common process smooths the initial surface by vigorous brooming which results in a value of n between those mentioned in *A*. Both views in this plate are in lower Rio Grande Valley, Tex.

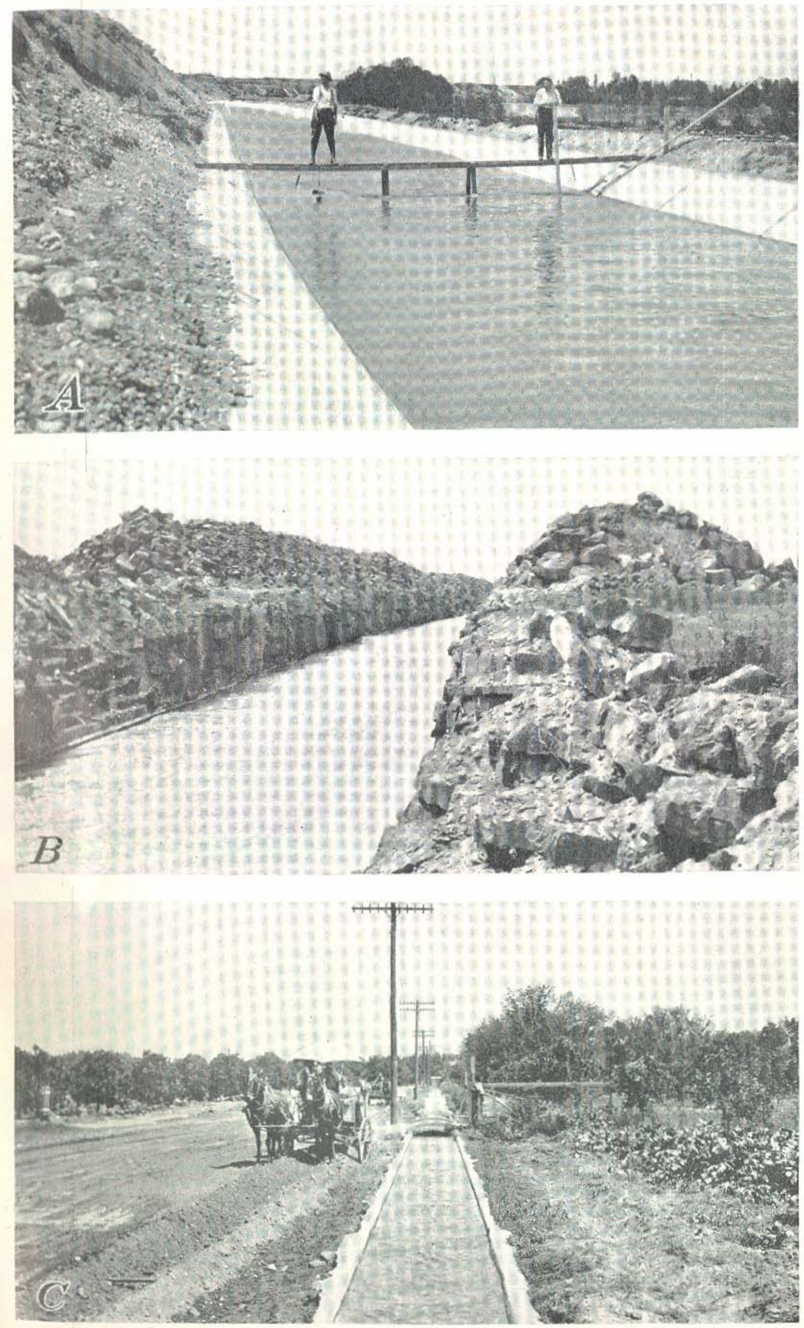


A, Floor and wall of Los Angeles aqueduct (from Owens River), Freeman division, after nearly 20 years of operation. (Nos. 18 to 24, inclusive.) *B*, Same aqueduct as shown in *A*, after rehabilitation, Mojave division. (Nos. 25 to 32, inclusive.) Very smooth floor of "3½ to 4½ hardness" (p. 15) after 2½ years of service when photographed in 1934. Note the test scratches in middle of view. Painstaking work in attaining this very smooth surface (n=0.012±) warranted if it results in the desired capacity and can be retained by reasonable maintenance expense. For long life, hardness is a requisite. *C*, Shooting concrete against a vertical back form. Note surface in foreground before applying a smooth troweled coat. Both primary and secondary roughness evident (p. 64). Views *A* and *B* by courtesy of Los Angeles Department of Water and Power.



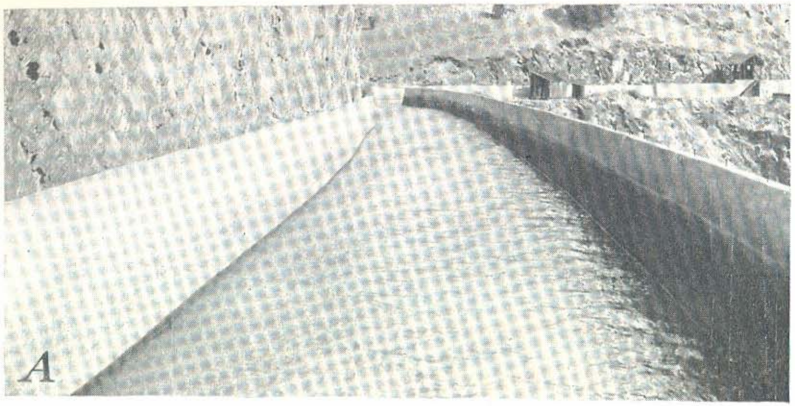
TYPICAL CONCRETE SURFACES.

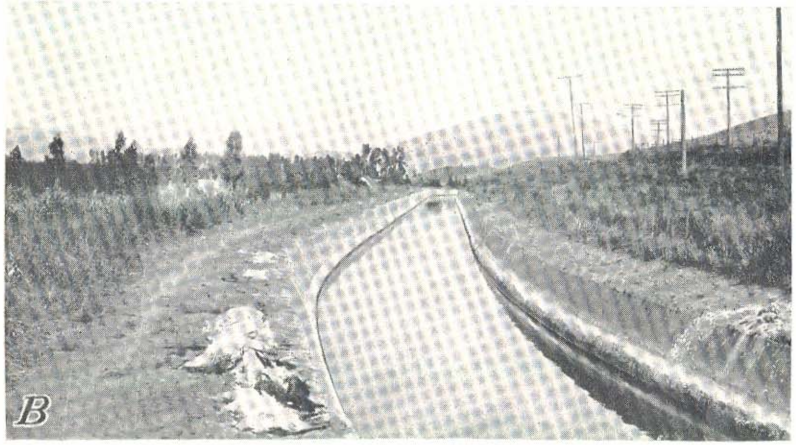
A, Very smooth concrete discounted by excessive deposits of caddis fly cases. *B*, Concrete with both primary and secondary roughness. Local surface is much rougher than is acceptable under modern specifications; also the obvious undulations further decrease the capacity of the channel. *C*, Slimes accumulate from some waters but may be removed, as shown by the dark surface to the right of the scale, restoring the original surface. *D*, High velocities accompanied by erosive material have removed the fines in the side wall above the watch, clearly outlining each pebble in the original aggregate.

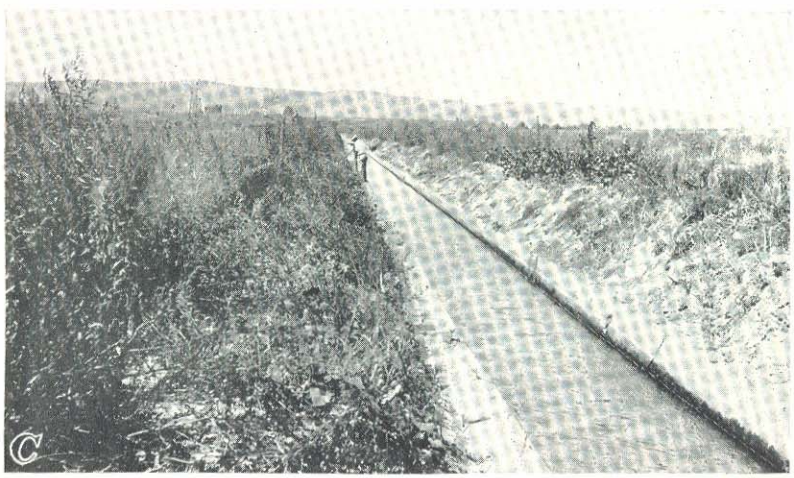


A, Ridenbaugh canal, Idaho. (Nos. 2 and 3; upstream from station 15.) Inspection in 1934 showed this lining still in excellent condition after nearly 25 years of service. *B*, North Side Twin Falls canal, Idaho. Note top of lining above water line. (No. 40; downstream from station 0.) *C*, Sanderfer ditch, California. (No. 62; upstream from station 3.) This canal has been placed in a covered pipe line.

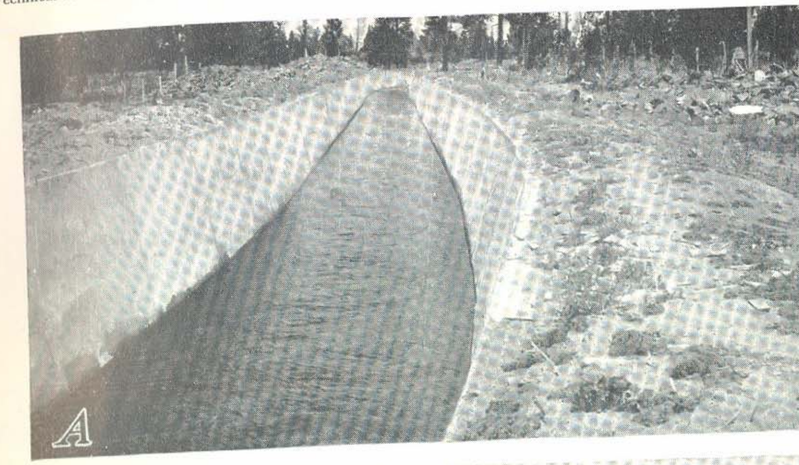
p.35



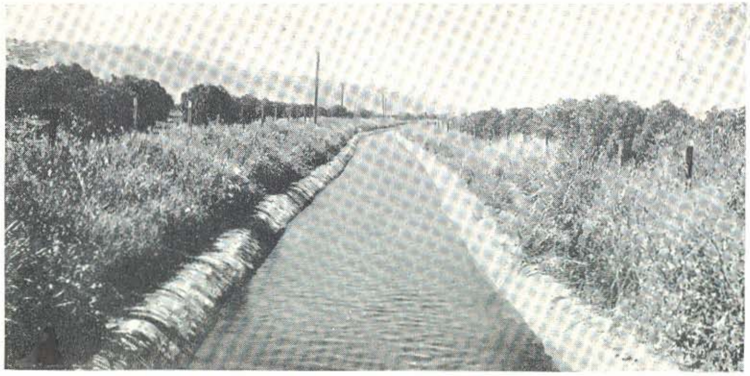


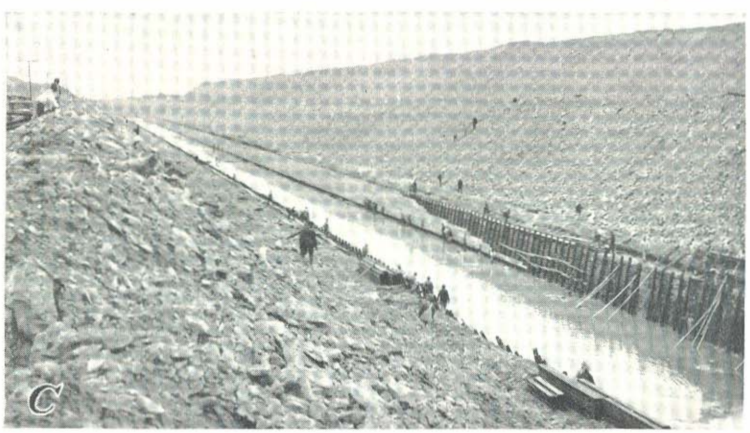


A, Modesto irrigation district main canal, California. (No. 91; upstream from station 3 plus 64, on meter bridge.) *B*, Santa Ana and Orange canal, Calif. (No. 92; upstream from station 12.) Note deposit below highwater line. *C*, Arroyo ditch, Calif. (No. 122; upstream from station 8.)

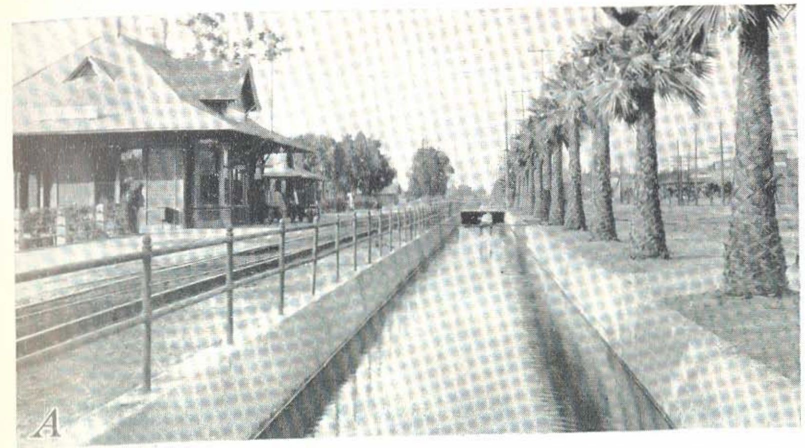


A, North canal, Bend, Oreg. (Nos. 123 to 128, inclusive; downstream from station 5.) *B*, Small ditch near Whittier, Calif. (No. 130; view upstream from station 2.) Clear water filling concrete section. *C*, Lower canal, Riverside, Calif. (No. 131; upstream from station 2.) A cement-plaster lining. Note weeds in broken places.

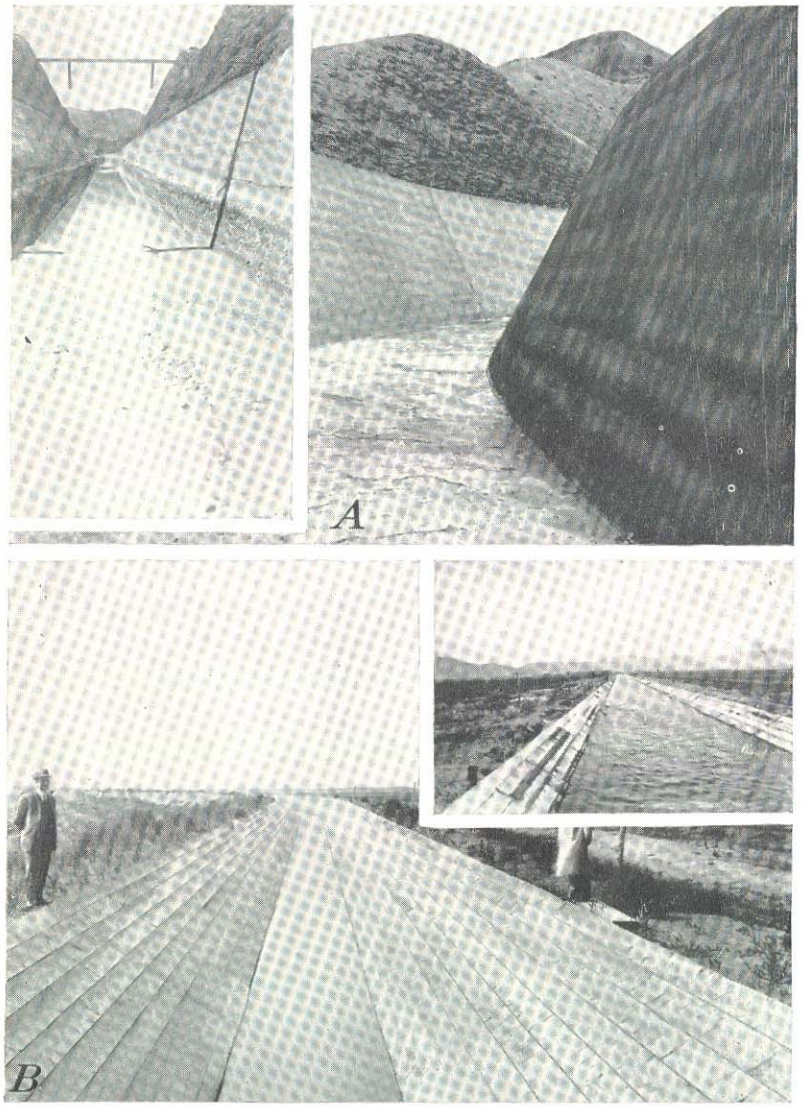




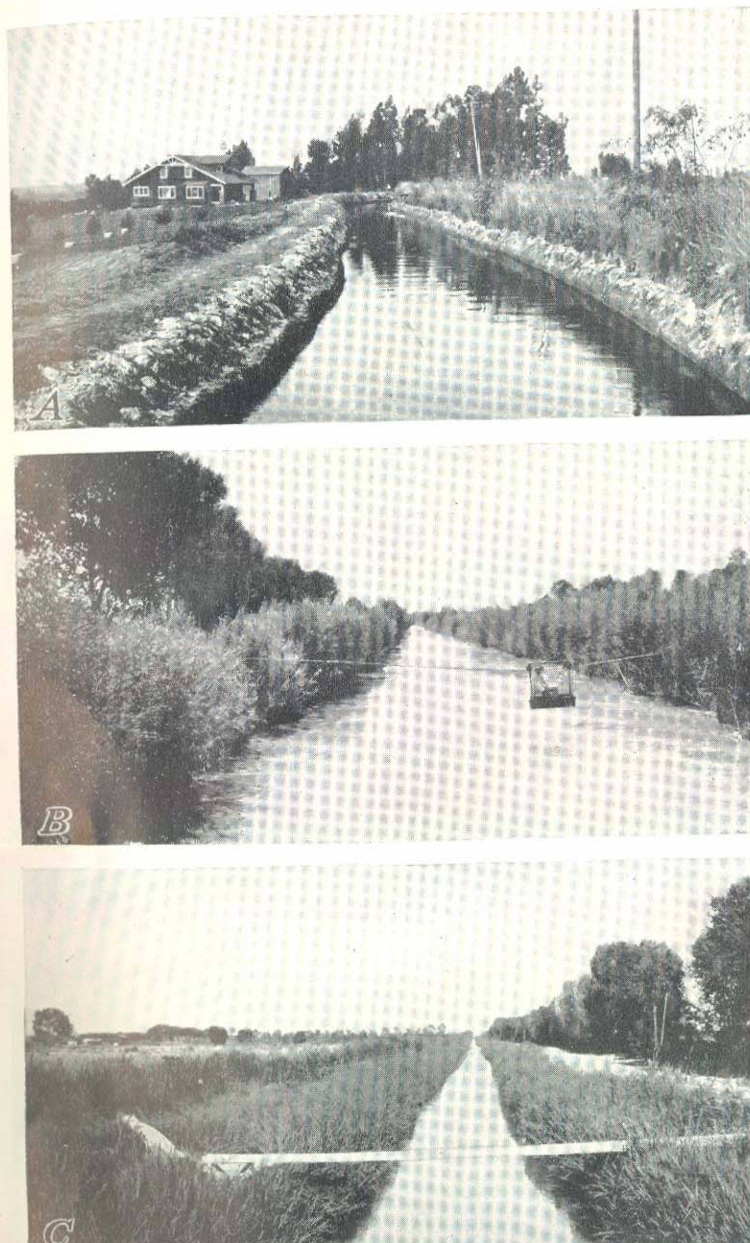
A, Lower canal, Lindsay-Strathmore irrigation district, California. (No. 169.) Concrete lining left as shot from a gun. Note characteristic uneven water line and concrete surface. Contrast this lining with later shot-concrete linings as made in Texas (pl. 2, *A*). *B*, Sulphur Creek wasteway, Washington. Note long standing swells. (See Nos. 435 to 450.) *C*, Maverick County Water Control and Improvement District No. 1, Texas. Excavated in thin strata (see rocks in foreground), the bottom was smooth but the sides of this canal were exceedingly rough. To increase the capacity the sides were lined with redwood planks spaced to allow water to pass freely and not build up an unbalanced static head on either side of the lining. In right foreground the vertical studding is shown anchored to the bed.



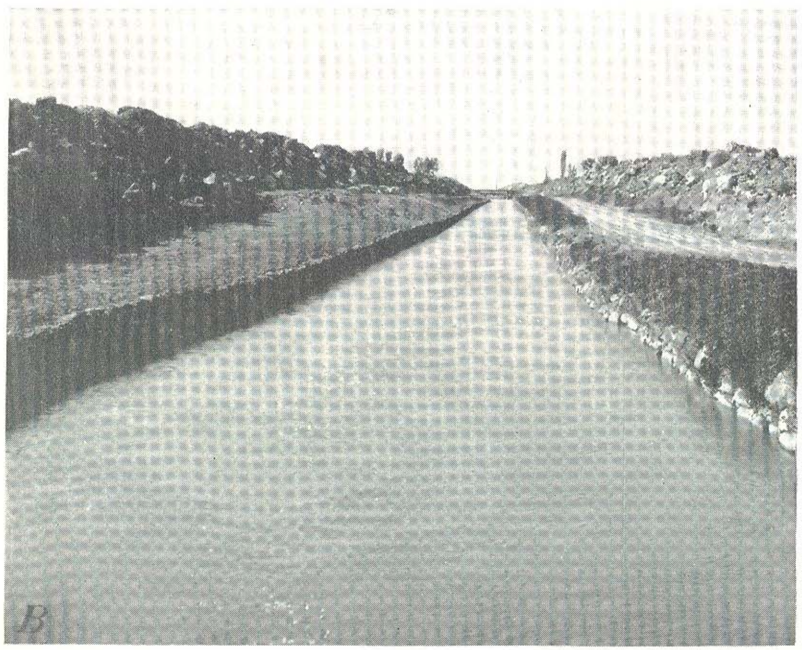
A, Upper canal, Riverside, Calif. (No. 132; upstream from station 3 plus 30.) Note meter station. *B*, Jacobs ditch, Boise, Idaho. (No. 431; downstream from station 0.) Rubble masonry, smoothly plastered walls, concrete bottom. *C*, Jacobs ditch, Boise, Idaho. (No. 432; upstream from station 1.) Unchinked rubble sides and bottom.



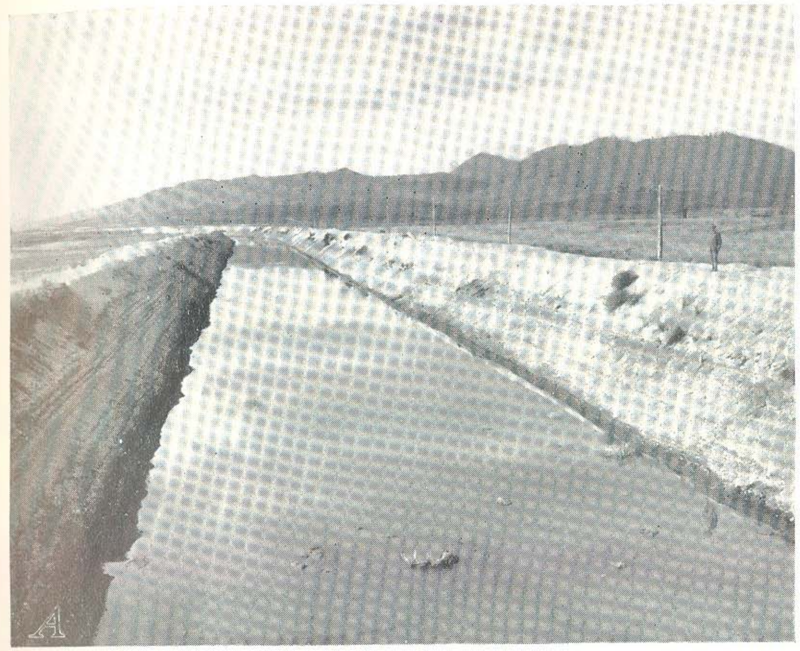
A, South canal, Uncompahgre project, Bureau of Reclamation, Colorado. Right-hand view shows lining when new. Left-hand view taken after some 20 years of use. Note erosion of lower part of side wall up to the usual high-water line. Current-meter gaging bridge causes shadow. (See Nos. 451 to 464.) The chutes are in similar channels. *B*, C canal, Klamath project, Bureau of Reclamation, Oregon. A timber-lined canal when new and (in inset) after some years of service. This canal was lined with concrete about 1921, replacing the timbers shown. Tests on the concrete section listed as Nos. 38 and 39. Bureau of Reclamation Photographs.

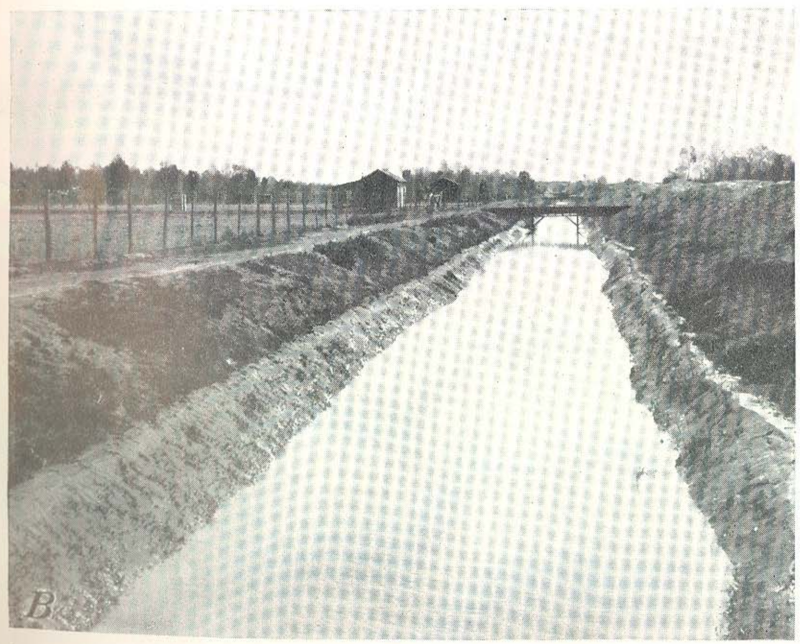


A, Santa Ana and Orange canal, California, just after being cleaned. (No. 257; upstream from station 10.) *B*, Central main canal, Imperial Valley, near Calexico, Calif. (No. 258; downstream. Portable gaging car shown over station 0.) When inspected in 1938 this reach of canal was completely free of brush and weeds as shown. Modern canal maintenance is conducive to better capacity conditions. *C*, Beech canal, Imperial irrigation district, California. (No. 315; upstream from meter station 10.) The muddy waters discourage aquatic growth in the water prism, but the fertile silt banks grow dense vegetation that drags down into the water prism and develops a high value of n.

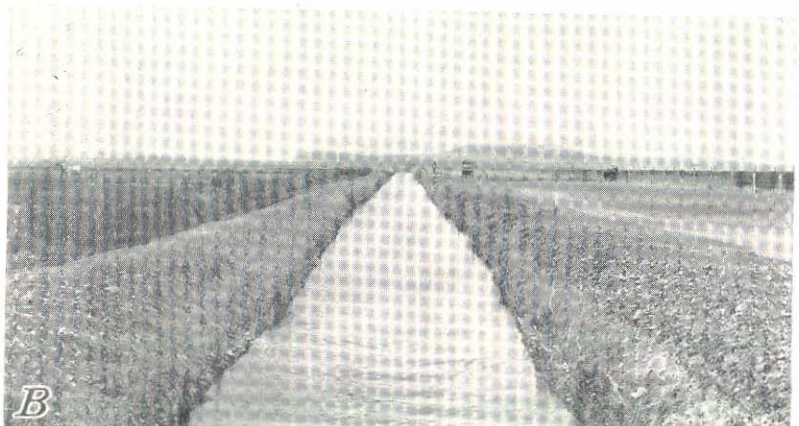


A, Canale Fossa di Pozzolo, Italy. (No. 401-2, viewed upstream.) An old canal with typical elliptical bed. The side on the left, approaching the vertical, is characteristic of old canals such as this in the United States. The other bank is a vertical wall. Note the rocky bed and the moss patches in the foreground. Photograph by courtesy of Marco Visentini who reported experiments on this canal. *B*, Milner-Gooding main canal, Idaho. Unusual construction. The dry-laid rock walls were backfilled with gravel and earth, through a badly fissured rock cut. Photograph from Bureau of Reclamation.

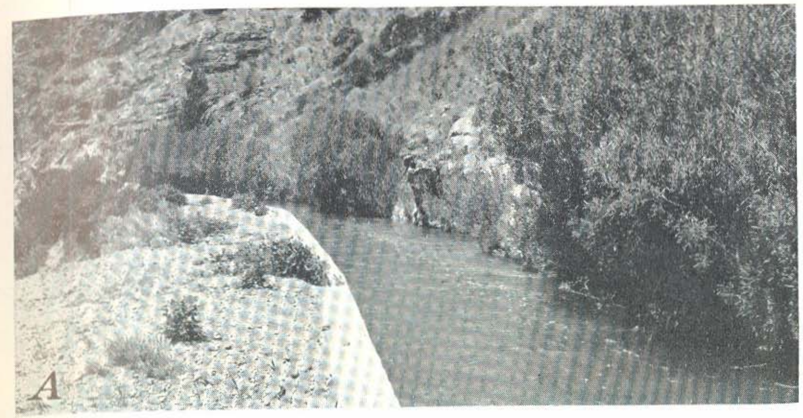




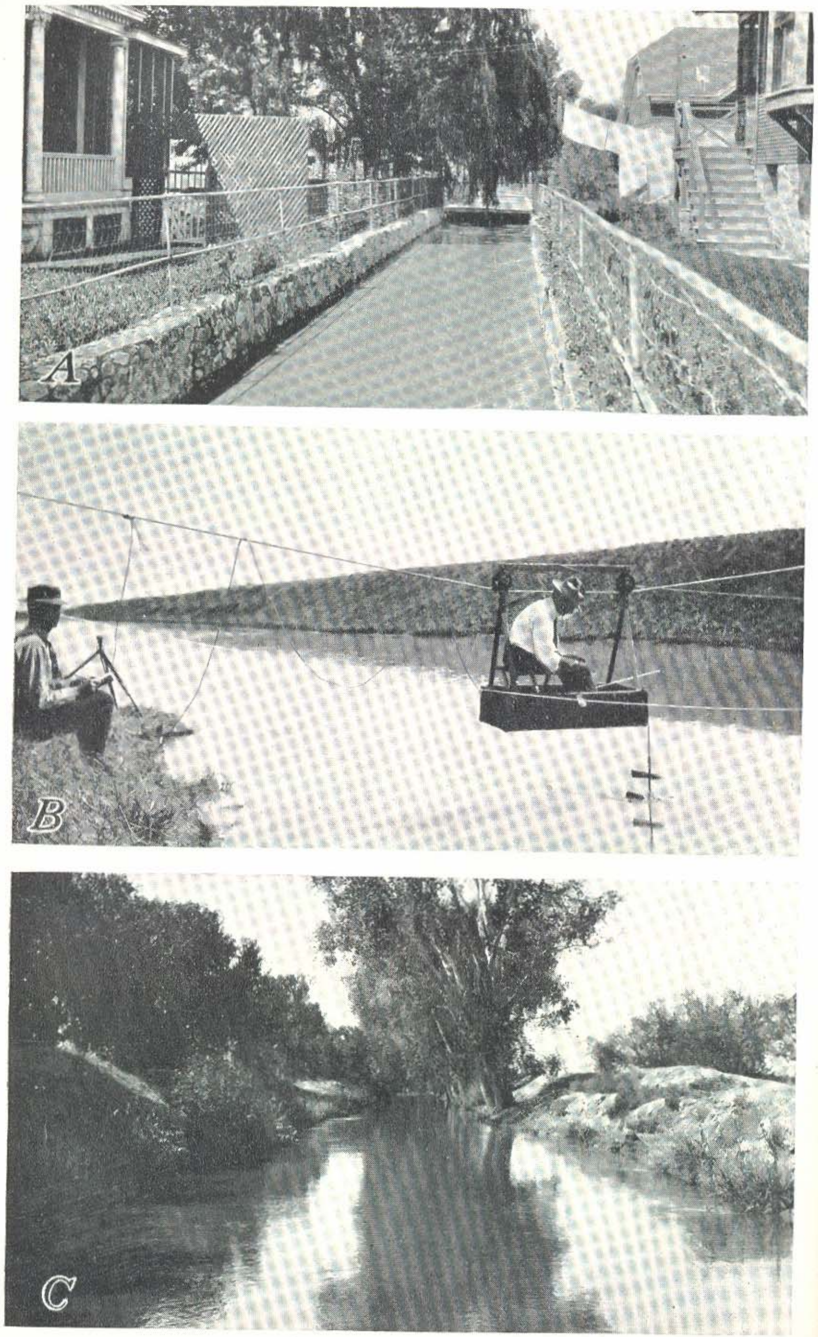
A, A new canal conveying clear water. The original trapezoidal shape beginning to round off at the corners to form the eventual ellipse (with nearly vertical sides if grasses root along the bank). *B*, A new canal or one recently cleaned, conveying silty waters. The mud berms develop at the sides and crowd in until an approximation to a regime channel is reached. Sometimes these berms are 2 feet or more wide while not more than a tenth of a foot or so has deposited on the bottom. Original material erodible at velocities of 3 feet per second becomes protected with a slick coat that will withstand velocities of 5 to 6 feet per second. Photographs from Bureau of Reclamation.



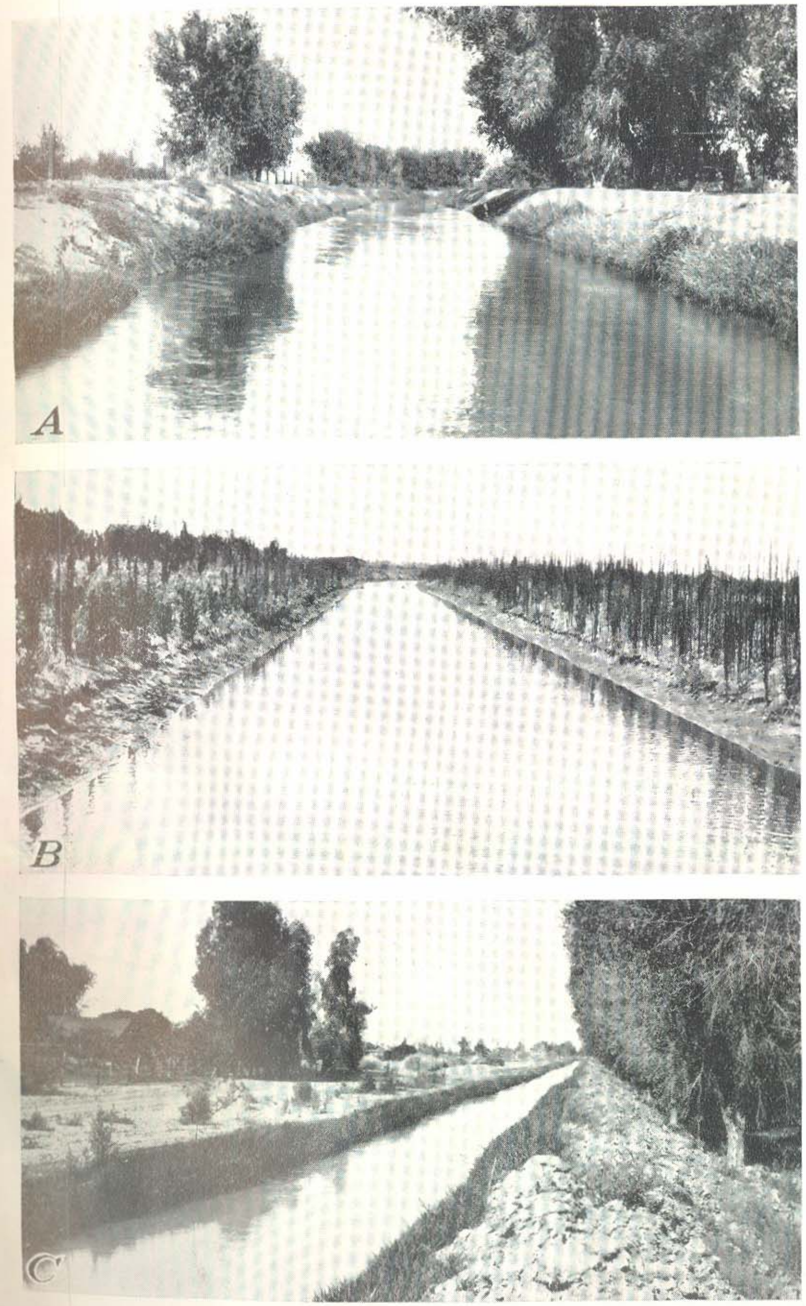
A, Salt River Valley canal, near Phoenix, Ariz. (No. 260; downstream from station 5.) B, Lateral 10, Orland project, Bureau of Reclamation, California. (No. 264; upstream from station 5.) C, River branch canal, Sacramento Valley, Calif. (No. 292; upstream from station 6. Station 0 below first bridge in distance.)



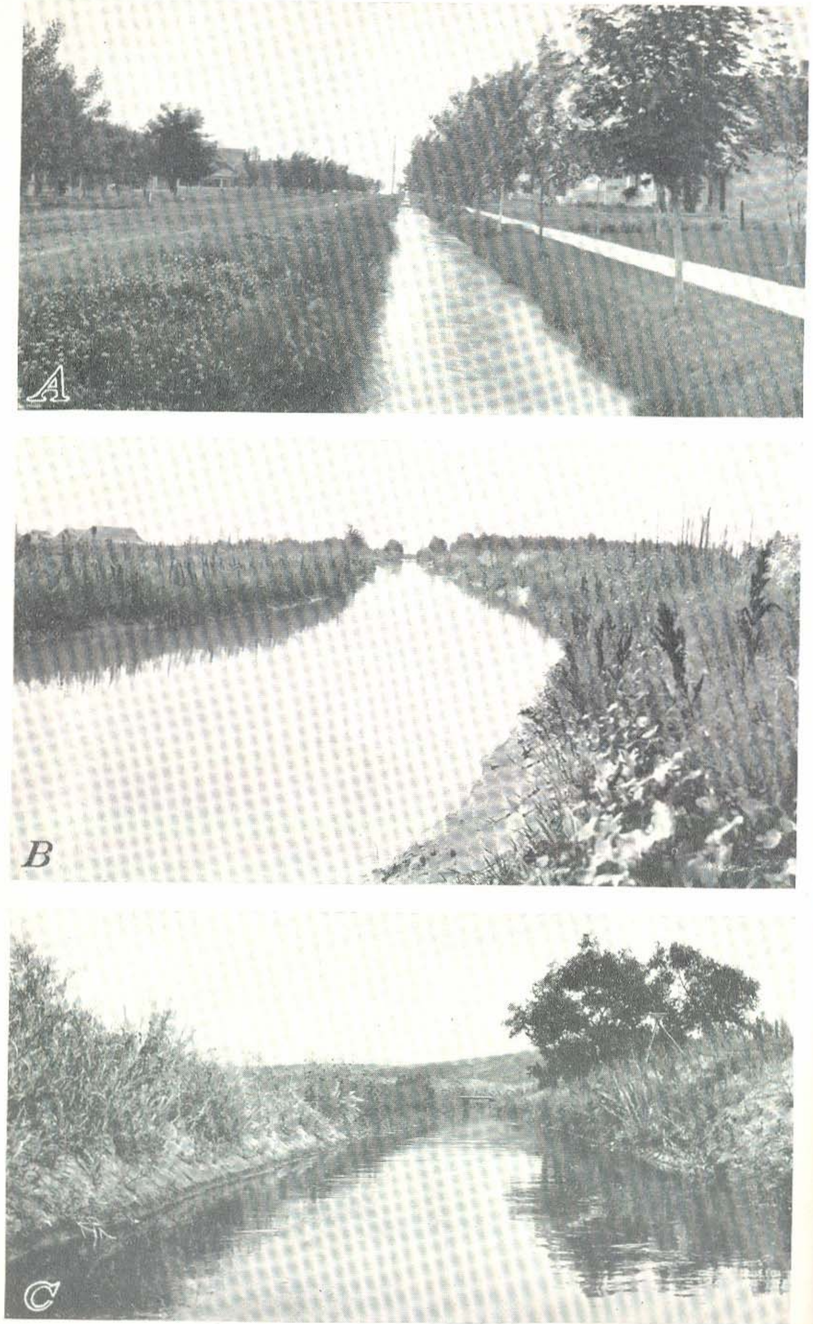
A, Logan, Hyde Park, and Smithfield canal, Utah. (No. 398; downstream near lower end of reach.) B, Lower canal, Riverside, Calif. (No. 405; upstream from station 6.) Note plank lining. C, Logan, Hyde Park, and Smithfield canal, Utah. (No. 408; downstream past meter station.) Note engineer's level in foreground, left.



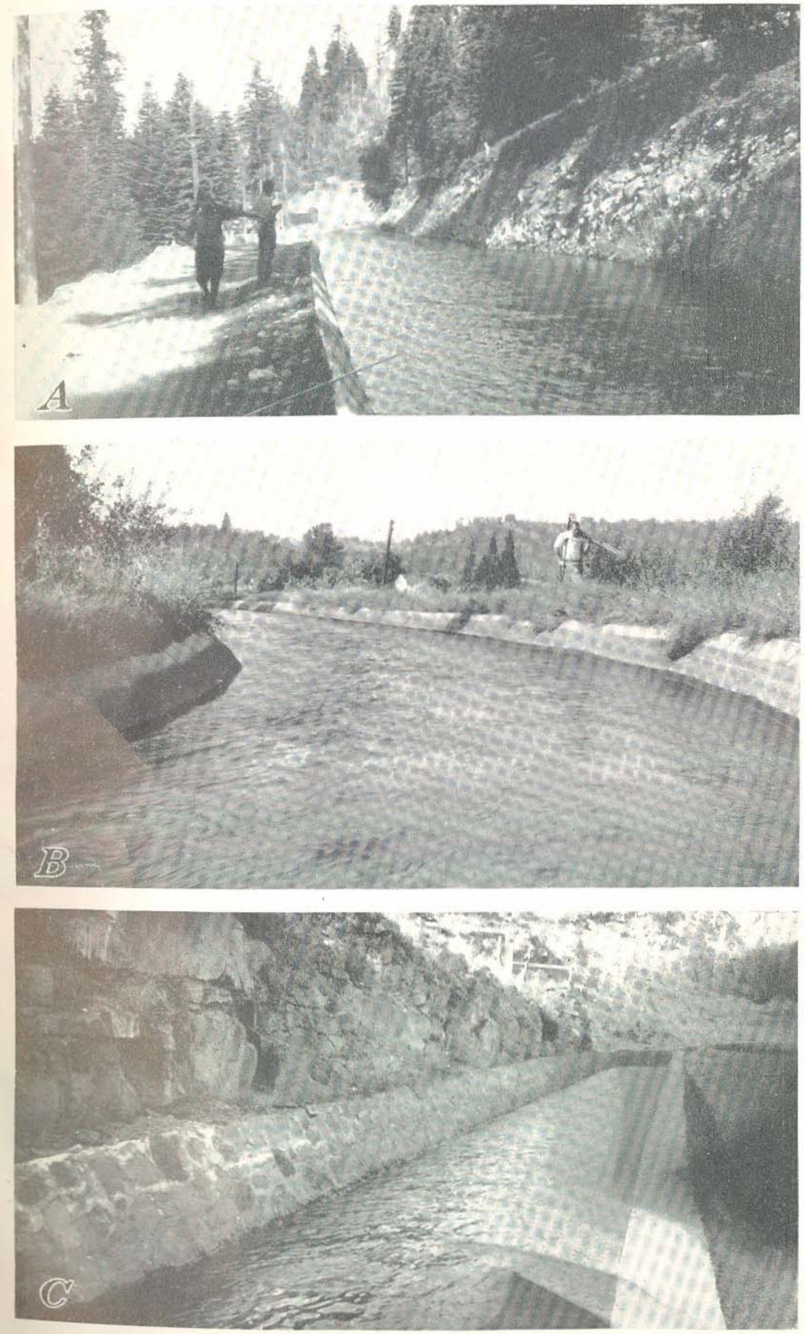
A, Orr ditch, Reno, Nev. (No. 434; upstream from station 1 plus 30. Station 0 is at bridge shown.) *B*, Farmer's (tri-State) canal, Nebraska. (No. 191; portable gaging car and meter station.) *C*, Maricopa canal, near Phoenix, Ariz. (No. 194; downstream from station 0.)



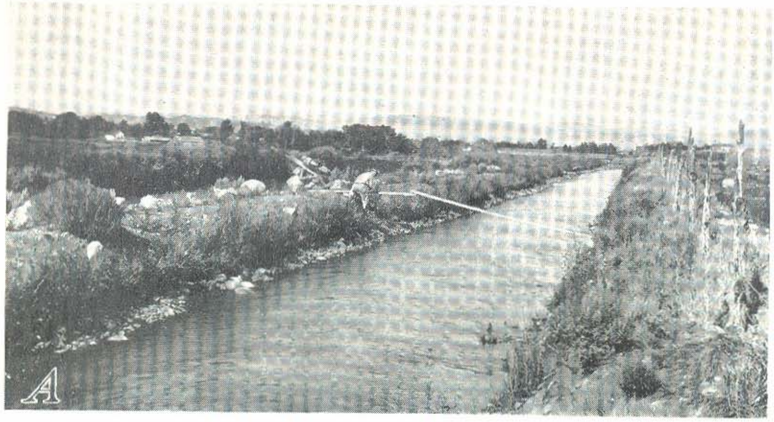
A, Grand canal, near Phoenix, Ariz. (No. 215; upstream from station 10.) *B*, Lateral 7, Turlock irrigation district, California. (No. 239; downstream from station 0.) Note general high-water line. *C*, Birch canal, Imperial Valley, Calif. (No. 250; downstream from meter station 6.)

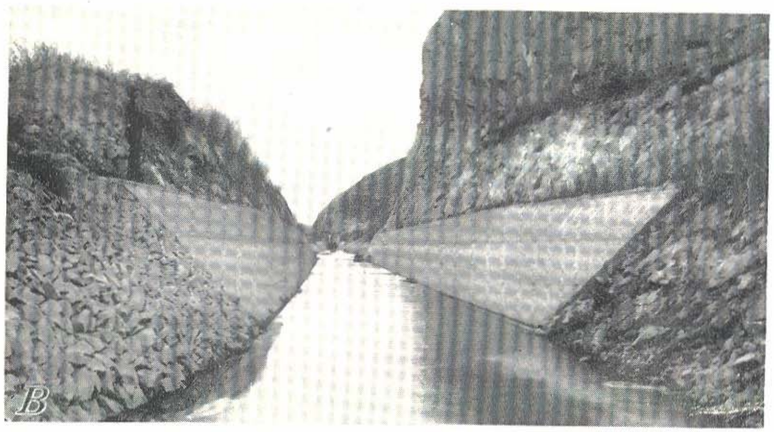


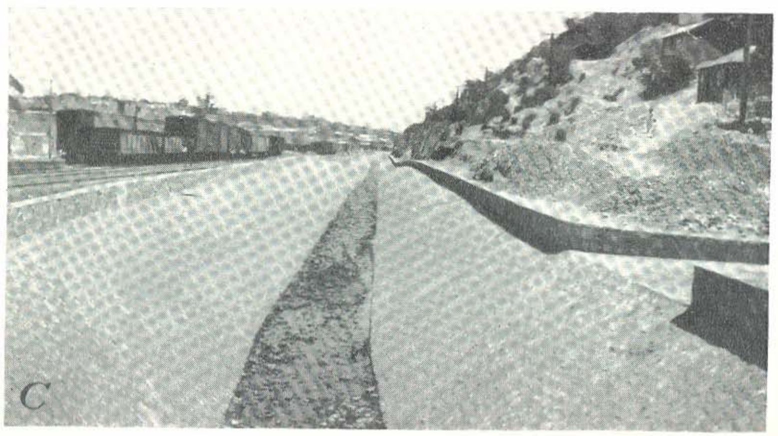
A, Lateral, South Side Twin Falls canal, Idaho. (No. 305; downstream from station 0.) *B*, Main branch, Turlock irrigation district, California. (No. 303; upstream from station 8.) *C*, Fullerton ditch, California. (No. 307; upstream from station 5.)



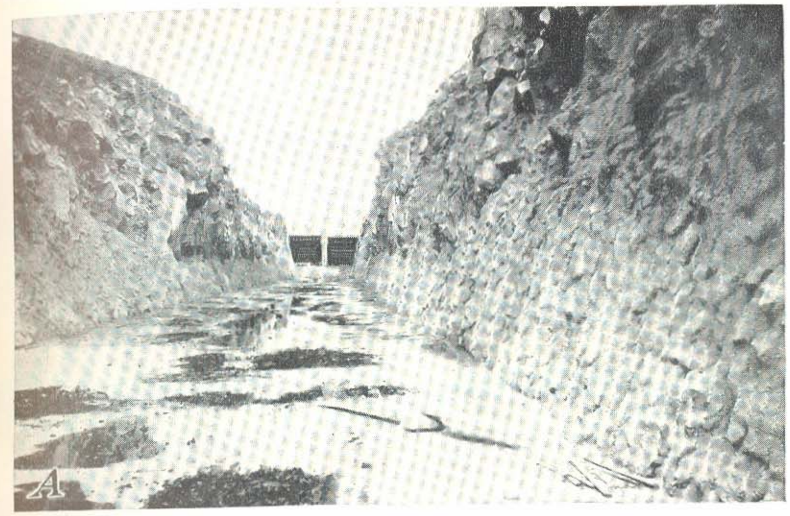
A, Drum canal, Pacific Gas & Electric Co., California. (No. 426.) Lower bank lined with concrete "plank," precast and placed where needed to prevent erosion. *B*, South canal, near Auburn, Calif. (No. 98; downstream from about station 7 plus 50.) Many sharp bends. Note highest velocity on inside of curve. Water on outside boils up and is uncertain in direction. *C*, Deschutes municipal district, near Bend, Oreg. (No. 428; downstream from station 2.) Masonry walls and concrete floor.



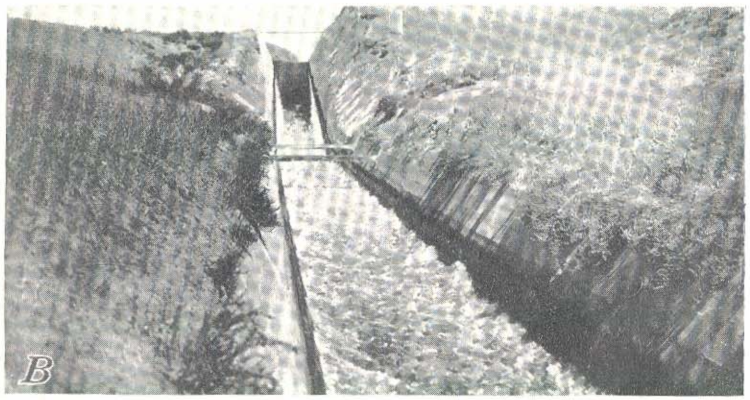


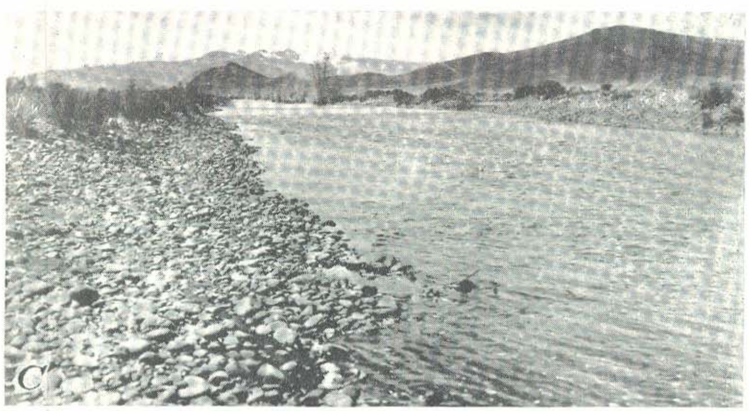


A, Reno ditch, Nevada. (No. 382; downstream from station 0.) *B*, The banks below a concrete section should be protected with riprap while the velocity is being reduced to that permissible in the earth section. Bureau of Reclamation photograph. *C*, Nogales floodway, near international boundary between Arizona and Sonora. The Mexican mason is very adept at this momposteria or rubble masonry. Sometimes such a wall is shot with concrete and broomed or troweled to a smoother finish. Photograph by engineers of the United States section, International Boundary Commission.

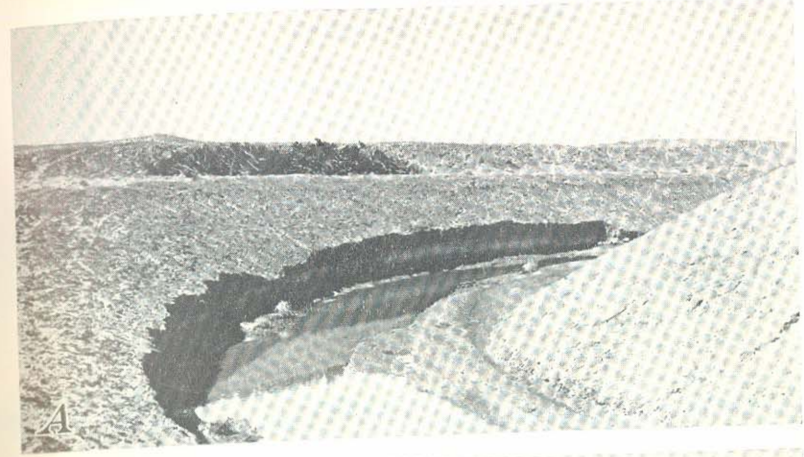


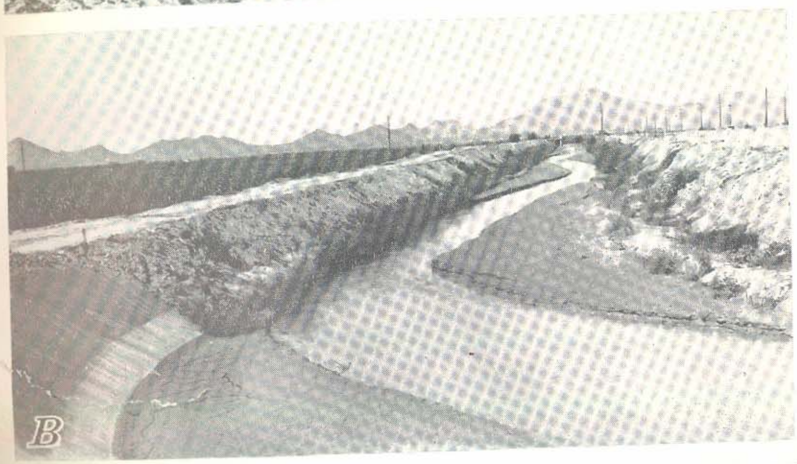
A, A very rough and porous cut in lava rock is made smooth on the bottom by a concrete lining while the sides are made watertight and materially smoother than before, with shot concrete. A good place to use Horton's formula for canal sections with different bed and side characteristics. Bureau of Reclamation photograph. *B*, Cachapoal canal, Braden Copper Co., Chile. (Nos. 349 to 370, inclusive.) Cobblestones are more in evidence on this canal than on canals in western United States that develop a cobble bottom. Other views submitted by the company showed large crews of men removing the loose cobbles from the bed with a skip and hoist. Note that values of n above 0.030 predominate. View by courtesy of Braden Copper Co.



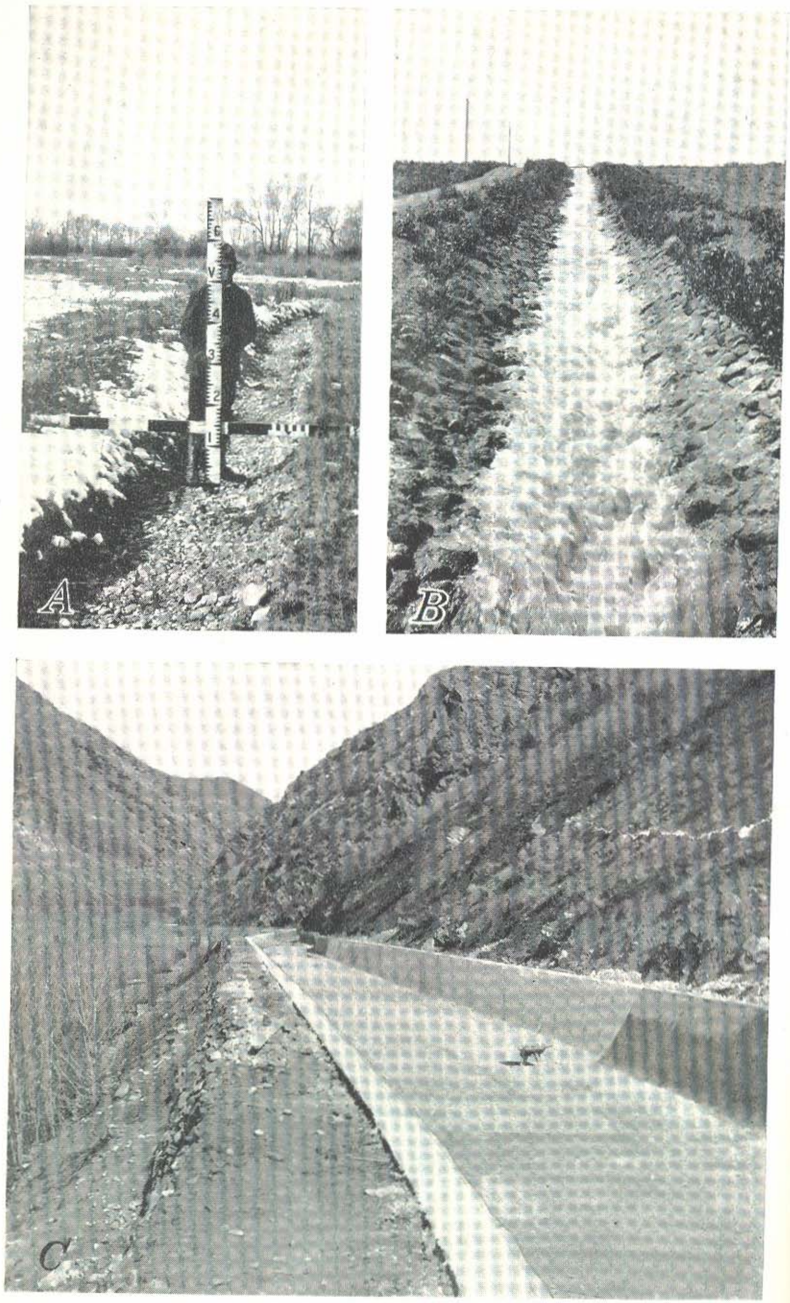


A, In chute channels, slug flow sometimes develops as shown and sometimes as rhythmic accretions not so clearly defined. The latter most often occurs near maximum capacity, the former at minimum flows. *B*, Chute flow in south canal, Uncompahgre project, Colorado. Note dark solid water as it pours over the brink at the top. Air enters the prism from the sides until the water is quite white at the bottom of the steepest part of the incline. The canal in the foreground is still steep enough to maintain shooting flow for the distance shown. In some of the chutes on this canal the hydraulic jump takes place a short distance below the incline. *C*, Rio Grande canal, San Luis Valley, Colo. (No. 348 on this canal but not this flow.) A typical cobble-bottom channel, where there is insufficient silt in the water, or too high a velocity, preventing formation of a graded smooth bed.





A, A new canal in unprotected sharp curves may erode the curve on the outside and deposit silt on the inside. Curves such as this may be protected with rock or brush riprap. *B*, Sometimes the designed capacity of a canal is not required for several years. If silty waters are the rule, something approaching a regime channel is developed by nature, inside the larger one. *C*, Canals conveying heavy silt and sand develop a characteristic bed, with deep depressions and dunelike shoals. This canal, in Imperial Valley, Calif., has the same bottom characteristics as the Fort Lyons canal, Colorado. This illustrates how a design shape, say of trapezoidal form, may be materially altered in operation.



A, Typical small cobble-bottom ditch. With the erosion of the "fines" in such terrain, a stable canal of low capacity is developed. Near Taos, N. Mex., small ditches several hundred years old are protected from continued erosion by the cobble. B, A steeply inclined canal can be given the characteristics of a mountain stream by a rough lining of grouted rocks. C, This part of the Davis and Weber Counties canal, Utah, has almost the characteristics of a bench flume. (Nos. 43 to 45.) Photographs A and B from Bureau of Reclamation.

DESCRIPTION OF CANALS

The descriptions in the following pages are supplementary to table 1, which gives all the information necessary to a clear understanding of the hydraulic conditions holding at various tests with the exception of a detailed description of the channel. The descriptions follow the same order and are numbered like those in the table. Missing numbers indicate there is no information additional to that found in table 1. The tense used is of the time the experiments were made.

CONCRETE LININGS

POURED OR HAND-LAID CONCRETE

No. 1, experiment DHB–4, Ridenbaugh canal, Nampa-Meridian irrigation district, Idaho. Test was made 2 years before and covered about the same reach as test No. 3 below. Bark found a slightly lower value for slope than later experimenters, which accounts for the lower value of n found. The preponderance of evidence indicates that a value of about 0.0125 is right for this excellent concrete, to include both tangents and curves. Coefficient $n=0.0110$.

No. 2, experiment FCS–24. Same canal as No. 1. Very smooth, hand-troweled, cement wash on a base of concrete 3½ inches thick. The reach is on tangent with about a 6° curve beginning at station 9. Lining was slabs 16 feet long with iron dowels and strips of tarred paper between slabs. After the forms were removed, joints were poured with a neat cement. As a rule the joints are as smooth to the hand as any other part of the lining (pl. 5, A), though slight cracks are opened during cold parts of the day. This is an exceptionally well-made lining, which, coupled with the fact that the curves are spiraled into the tangents, accounts for the very low value of n found by all experimenters. For additional experience on this canal see Nos. 1 to 15, table 1. Examination 20 years later showed little deterioration in this concrete. Coefficient $n=0.0121$.[12]

No. 3, experiment FCS–24a, was on the same canal as FCS–24, but the reach included not only the 901 feet of tangent as above, but also the above-mentioned curve, which was about 600 feet long, and a short reach of tangent below the curve, making the total reach 1,819 feet long. As is to be expected, the value of n is a little higher than on tangent. The slabs on curves were but 12 feet long. Coefficient $n=0.0129$.

No. 4, experiment BPF–3. This experiment was made on approximately the same reach of canal as FCS–24, but was 1,020.6 feet long, with one slight curve in the reach. The slopes of the surface in this and experiments 44 and 45 were found by a line of levels run between the ends of reaches as usual, but the water surface was found by means of a gage constructed on the piezometric principle. The slope given in the table is the mean of 23 tests. Coefficient $n=0.0124$.

Nos. 16 and 17, experiments JBL–6 and 5. Former supply conduit for Los Angeles, Calif. Covered conduit. In use 4 years, one curve in section tested. Where wetted, section was very smooth. Apparently of 1 to 3 cement-mortar plaster on concrete. No deposit or growth. Coefficients $n=0.0108$ and $n=0.0111$.

Nos. 18 to 32, inclusive, experiments RRP. Los Angeles aqueduct, Calif. These tests comprised a most comprehensive set of experiments, made to determine possibilities of increasing the capacity of the aqueduct from Haiwee Reservoir to San Fernando Valley. This conduit is used for municipal supply, power, and irrigation. Concrete-lined sections only were studied for this bulletin. Field and office experiments and computations by forces of the department of water and power, under the immediate direction of R. R. Proctor, field engineer. Two typical reaches are included in table 1 data. (See below.) For both, the discharge measurements were made at both ends of the reach by current meter, held at 0.2, 0.4, 0.6, and 0.8 of the water depth in six verticals. The computations for Q were made by using both vertical and horizontal velocity curves. The results are stated to be within 0.5 percent of the true discharge. The observed loss of water was prorated according to distance below the initial measurement. For the purpose of the original experiments the method of study and determina-

[12] Additional tests on this and other canals in the Boise Valley will be found in (64) which is excerpted from an unpublished report by W. G. Steward entitled "The Determination of n in Kutter's Formula for Various Canals, Flumes, and Chutes on the Boise Project and Vicinity." 1913.

tion of values of Kutter's n are given on page 13. The values of the roughness coefficients given in table 1, are based on slope of the energy gradient and weighted values of the average velocities and hydraulic radii throughout the reaches as listed, using cross-sectional dimensions for many intermediate stations.

Nos. 18 to 24, inclusive, experiment RRP, Freeman division. General methods are outlined in the paragraph above. The lining in this division was found to be in such good condition after some 15 years of operation that practically no repairs were made (pl. 3, *A*). The fact that a general value of $n=0.0125$ was found on a long reach designed with $n=0.014$ supported the plan to bring all of the concrete-lined portion of the aqueduct to this standard or better and also established the fact that a smooth surface of great hardness not only could be constructed but also could be relied upon to maintain its desired smoothness, whereas other localities showed that inferior concrete lining, while perhaps originally fairly smooth, was too soft to withstand the eroding effect of abrasive matter entering nearly all conduits.

Nos. 25 to 32, inclusive, experiment RRP, Mohave division. General methods are outlined in the paragraphs above. This division included long reaches that required improvement to convey the desired flow of water (pl. 3, *B*). The tests listed as Nos. 25 to 27, inclusive, took place before any repairs were made, and the later tests after improvement. The change in values of n is attributed entirely to the smoother bottom installed after the preceding tests.

No. 33, experiment FCS–98, small lateral, 13 miles north of Weslaco, Tex. Hand-laid and troweled concrete. V-shaped with bottom rounded on 10-inch radius. Free of mud, sand, and moss. 15-mile wind downstream. Swept clean just before test, hence conveyance of muddy Rio Grande water not in point. Coefficient $n=0.0133$.

Nos. 38 and 39, experiments BR–D, South Branch (C) canal, Klamath project, Bureau of Reclamation, Oreg.[13] Tests by Allan Darr on 3-inch concrete lining poured in 1919–20 to replace former timber lining on earth fill. Methods of tests based on those given as laid down by the author (*55*). Sections taken every 100 feet. Coefficient $n=0.0135$ and 0.0126.

No. 40, experiment FCS–19, North Side Twin Falls Land & Water Co.'s main canal near Milner, Idaho. As shown in plate 5, *B*, this concrete lining fills out the main irregularities in a very rough lava-rock cut. An examination of the section below the water line was impossible at the time of making the experiment, and the various cross sections from which the value of *R* was deduced were taken from office notes. A study of these notes shows that the bottom is undulating and that while the high velocity would prevent the accumulation of sand deposits, the velocity is slightly retarded by the disturbance in the filaments of current due to the undulations. Coefficient $n=0.0138$.

No. 41, experiment BR–F, Carlsbad project, Bureau of Reclamation, N. Mex. Tests by C. A. May and E. C. Koppen, October 1915; after greatest demand. Gravel and weeds typical of time of season. Concrete hand-finished, true to section and grade. Expansion joints, asphalt strips, at 50-foot intervals, projecting one-fourth to one-half inch. Five percent of bottom covered with gravel from fine to 2-inch diameter. Alignment: First 246 feet on 8° curve, thence 500 feet tangent, thence 250 feet 8° curve. Metered at midsection. Levels between nail heads in top of stakes. Depths by level and rod. Coefficient $n=0.0137$.

No. 42, experiment BR–F, 1 mile below No. 41. One slight curve. Lining and gravel just like in No. 41 test. In addition, slight retardation caused by weeds dragging on surface near the edges. Coefficient $n=0.0139$.

No. 43, experiment FCS–13, Davis and Weber Counties canal, Utah (pl. 24, *C*). An example of the retarding effect of wooden expansion joints if they project into the canal section. Lining was laid in slabs from 8 to 16 feet in width. Strips of wood a little larger than building lath were placed between slabs with the idea that they would eventually be pulled and the space filled with asphalt. This was not done, so strips project from 0 to 1½ inches into the section. Likewise, velocity at the bottom was retarded by small patches of gravel, sloughed off the hillside cut in which the canal runs. Coefficient $n=0.0154$.

No. 44, experiment BPF–1, Davis and Weber Counties canal, Utah. This experiment was in the same canal as tests Nos. 43 and 45, but about 8 miles upstream and about 1 mile below the head gate from the river. Condition of bottom could not be determined. The concrete on sides was smooth and unbroken. The hydraulic grade was taken as the mean of five tests with level and

[13] DARR, A. L. EXPERIMENTAL INVESTIGATIONS "C" CANAL, KLAMATH PROJECT, OREGON-CALIFORNIA. 26 pp., illus. 1922–23. [Unpublished.]

piezometer and found to be 0.000413, while constructed grade of this portion of canal was 0.000445. Coefficient $n=0.014$.

No. 45, experiment BPF–2. This test was on a reach 468.5 feet long, included in the 1,000 feet described in No. 44. Patches of gravel of all sizes up to 5 inches in greatest dimension covered 10 percent of the area, mostly adjoining the toes of the side slopes. Coefficient $n=0.0146$.

No. 60, experiment FCS–94, lateral C, near Edinburgh, Tex. Concrete in panels with projecting asphaltic joint fillers. Smoother than broomed shot concrete but not the equal of well-troweled surface. Coefficient $n=0.0141$.

No. 61, experiment DHB–10, King Hill canal, Idaho. Test was on both tangent and curves. The concrete was not surfaced, but left as hand tamped to grade. After surface coat had set, the 2- by 4-inch end forms were removed and the groove poured with a 1 to 1 mixture of sand and cement. The surface was very rough, especially at the joints. The canal was clean of detritus and moss. Coefficient $n=0.0143$.

No. 62, experiment FCS–69, Sanderfer Ditch Co.'s main canal, near Whittier, Calif. As shown in plate 5, *C*, this reach is straight and uniform. The bottom is slightly dished. As is the case of many small lined ditches in southern California, the sides and bottom are covered with a rough deposit which entirely vitiates the good results which would be anticipated by using a smooth cement wash such as the one on this ditch. This deposit appears to accumulate on either smooth or rough concrete, so the added expense of the former does not appear to be warranted in view of the results. The water in this ditch was clear and without sand. Since this test, the canal has been placed in a concrete pipe and covered. Coefficient $n=0.0155$.

Nos. 63–66, experiments BR–F–12 to 15, South canal, Uncompahgre project, Bureau of Reclamation, Colo.[14] Concrete cast against board forms about 1907. Experiments F–12 and 13 were on reaches that included frequent curves and short tangents. Lining had many cracks and the cross section was contracted by relining in several places. No. F–14 was on a tangent where original finish was a plaster coat of cement. The sides were still in good condition but the bottom was worn, cracked, and heaved in places. No. F–15 was on tangents with one slight curve in concrete cast against board forms and still in good condition. (See F–11 on chute, No. 453.) Foster's tests were about 3 years after Cone's (No. 67) and 16 years before Lane's (Nos. 454 to 464, inclusive). Lined portions of this canal were designed with $n=0.012$; after 8 years n varied between about 0.014 and 0.017 while 15 years later n had increased but little. Apparently concrete such as this, subject to high velocities with water containing much abrasive debris, finally attains a roughness corresponding to $n=0.018$ or less, then holds that value because the bottom velocities are retarded by the rough surface. Likewise all the fine cement, sand, and gravel have been washed out, at the surface, leaving fair-sized pebbles well-bedded in a cement matrix. This rough finish becomes the final operating surface.

No. 67, experiment VMC (C), same canal (pl. 10, *A*). Reach between miles 9 and 10. See above for descriptions. Canal carrying less than one-tenth rated capacity. Coefficient $n=0.0155$. Three years before Foster's tests above.

No. 68, experiment JBL–8, Santa Ana canal, near Yorba, Calif. Concrete tamped behind board forms. No plaster coat. Section wide and shallow. Several inches of sand in bottom. Moss and grass in patches on sides. Coefficient $n=0.0157$.

No. 87, experiment FCS–37a, lateral 12, Orland project, Bureau of Reclamation, California. A small lined section of trapezoidal form, with a slight dishing in the bottom. About 50 feet above station 0 is the lower end of a chute drop, and the ditch below station two turns to the right 90° in a curve of 34 feet radius. The surface of the channel was a good grade of concrete, but not smooth washed. It had a slight deposit of slimy silt, which would have allowed a low value of n but for gravel scattered throughout the ditch section. Coefficient $n=0.0160$.

No. 88, experiment FCS–37. On the same lateral as No. 87 but covers a straight reach immediately below the right-angle curve noted above. In the opinion of the author the value of n in this experiment is better for the gravel condition in a small lined section than that found in the shorter reach used for No. 87. This gravel ranged in size from fine to that of a walnut and had a marked influence in retarding the velocity, as there was more or less movement of the gravel down the

[14] LONGWELL, J. S. SOUTH CANAL. VALUE OF "N". U. S. Bur. Reclam. 19 pp., illus. 1917. [Unpublished report.]

channel, which retards velocity more than does stationary gravel. Coefficient $n=0.0192$.

No. 89, experiment JBL–7, Colton canal, near Colton, Calif. Lining of unplastered concrete. No sand or gravel. Sides and bottom covered with thin coat of moss. Coefficient $n=0.0167$.

No. 90, Experiment FCS–11, South Cottonwood Ward canal, near Murray, Utah. A reach was chosen in the middle of 450 feet of lined section between an earth section and a flume. A slight curve at the upper end. A deposit of about 0.07 foot of fine sand and rootlike growths covered the bottom and modified the original section of rather rough concrete. A slight deposit of moss and slime also modified the sides of the channel. Coefficient $n=0.0171$.

No. 91, experiment FCS–55, Modesto irrigation district main canal, near La Grange, Calif. As shown in plate 6, *A*, this reach of canal is on an approximate tangent. There is a very sharp curve about 50 feet below the reach tested. The lining is a fair grade of concrete, being about as rough as an orange. The value of *n* is high, because of the presence of a number of pieces of slate rock that have fallen into the canal from the adjoining cliffs. This influence probably is materially reduced when the canal is carrying water to capacity. However, this experiment shows the value of cleaning the canal as often as practicable in order to maintain a high carrying capacity which is much desired by this district. Coefficient $n=0.0174$.

No. 92, experiment FCS–63, Santa Ana and Orange canal, near Orange, Calif. In the reach tested, there was a gentle curve between stations 5 and 7. As shown in plate 6, *B*, taken from about 200 feet below station 10, this canal has the rough deposit and moss common to southern California ditches. In addition the concrete lining of the bottom has been completely covered by a deposit of soft sand from 0.1 to 0.2 foot deep. This lining had originally been a reasonably smooth piece of work, but the deposits had destroyed much of the usefulness of the smooth concrete. Coefficient $n=0.0176$.

Nos. 93–118, experiments FCS–101 to 117, inclusive, South canal near Auburn, Calif. This extended series of experiments was primarily for purpose of developing influence of extremely sharp bends in canals (pl. 19, *B*). This influence affects both the value of *n* and the position of water surface at two sides of the canal. The concrete surface was quite rough; a value of *n* in long tangents would have been about 0.016. Long reaches of varying total curvature were chosen. Careful current-meter measurements were made to determine discharge. The average water surface was taken with a special piezometer device, on both right and left banks of the canal, every 50 feet on tangents and oftener around bends. Plotting of the water surface on both edges told little as to the average slope of the canal. However, when the elevations at the two sides were averaged and to this mean elevation was added the velocity head for the mean velocity at that station, a point on the energy gradient was disclosed. The developed energy line was on a very even slope and practically smooth. The evenness of slope indicated that there was no extra loss of head concentrated at the sharp bends, but that the loss was distributed along the whole reach and the result was a higher value of *n* for the reaches of most curvature. This was the first time experimental data disclosed this fact, so far as is known to the author (*57*, p. *61*).

Nos. 119 and 120, experiments ES, Camuzzoni canal, Verona, Italy. An industrial canal of large size, constructed in 1896, in rough but even concrete. First tested when new by H. Bazin just before offering his 1897 formula. Coefficient 0.018 for No. 119. Again tested in 1924 by E. Scimemi (No. 120) and the coefficient found to be $n=0.022$.

No. 121, experiment FCS–70, Los Nietos Water Co.'s main canal, near Whittier, Calif. Original lining in this canal is fairly smooth, but the deposit common to this region has so changed its character that, aided by the rolling sand, a high value of *n* is found. This sand was about 0.03 foot deep. There was also a slight retarding effect due to grass and weeds dragging on the surface of the water near the edges of the channel. Coefficient $n=0.0188$.

No. 122, experiment FCS–67, Arroyo Ditch & Water Co.'s main canal, near Whittier, Calif. As shown in plate 6, *C*, this rough-finish concrete section has accumulated a deposit of rough mossy growth that greatly retards the velocity of the water. In a few places throughout the reach tested the lining was irregular and not in true alignment, which also tended to increase the value of *n*. The reach was on tangent, with a sharp angle about 50 feet above station 0. Coefficient $n=0.0188$.

Nos. 123 to 128, experiments FCS–31, FCS–30, FCS–32, Central Oregon Irrigation Co.'s north canal, near Bend, Oreg. These experiments were with varying discharges, on consecutive days, in identical reaches; (a) is on a tangent

240 feet long between a 15° curve above and a 14° curve below; (b) embraces 157 feet of tangent, then 154 feet of 14° curve to the right, then 90 feet tangent, then 109 feet of 15° curve to the left, then the tangent that includes (a) 240 feet long, then 178 feet of 14° curve.

This lining is clean-scoured, very rough, and deeply pitted concrete made in a rough lava-rock cut. As shown on plate 7, *A*, the cross sectional form is even and the filaments of current are not disturbed except by the curves. The inherent roughness of the lining accounts for the high values of *n*.

This lining was a 1 : 4 : 5 mixture, deposited behind shiplap forms against a hand-laid rock wall, filling the cavities in a rough rock cut. Expansion joints of ¼- by 4-inch lumber were placed on sides and bottom every 12 feet and left in the concrete.

No. 129, experiment JBL–4, upper canal, Riverside Water Co., California. Coat of 1 to 3 cement plaster roughly applied to concrete. Canal partially cleaned of a stringy grass a few days before test. Isolated bunches of grass left in bottom. Two curves included in reach. Coefficient $n=0.0218$.

No. 130, experiment FCS–68, small ditch from pumping plant, California. Although constructed with a smooth-finished cement wash, this ditch shows a high value of *n* because a dark, crinkly deposit has changed the condition of the walls. Vegetation on the banks dragged in the water and retarded velocity to a slight extent (pl. 7, *B*). This test is not given full weight because the ditch is too small to give a first-class current-meter measurement. The mean of three measurements was used. Coefficient $n=0.0220$.

No. 131, experiment FCS–75, Riverside Water Co.'s lower canal, Riverside, Calif. This experiment gives a good example of a cement-wash lining in which under favorable conditions in southern California a friction factor of about 0.018 might be expected without removing the sand which appears to be ever present in the canals in this vicinity. If the sand were removed by the addition of numerous sand sumps and gates this factor would be reduced to 0.016 or thereabouts. At the time of making the tests on this canal, the lining had been broken in scattered spots, allowing vegetation to root and grow as shown in plate 7, *C*. In the bottom of the channel were scattered deposits of loose sand, covering possibly 10 percent of the bottom area. In some of these deposits moss and water grasses flourished.

This lining was a cement and sand coat about 1 inch thick, applied directly to the trimmed surface of the earth channel. Occasional fractures in such a lining are to be expected. Coefficient $n=0.0221$.

No. 132, experiment FCS–71, Riverside Water Co.'s upper canal, in Riverside, Calif. While originally the canal was lined with a well-built and but lightly pitted cement-wash surface, the bottom of the channel has completely lost its identity as a concrete lining insofar as friction is concerned, since there is now more than 18 inches of sand in the bottom. This drifts down the canal in little pockets that look like hoof prints of livestock. The positions of these shift rapidly, causing the depth of water at a given point to change 0.4 or 0.5 foot in about 30 minutes. This condition renders a measurement by current meter using multiple points obviously inaccurate, hence the integration method was used, as the latter gives results as close to those found by multiple points as can be desired. A measurement by this method takes but a few minutes, and the canal bottom in this period probably does not shift sufficiently to vitiate the results. As shown in plate 9, *A*, there are no curves or structures above the reach tested to change results, and the same condition holds downstream. Coefficient $n=0.0231$.

SHOT-CONCRETE

No. 133, experiment FCS–91, Rossow canal, Hidalgo County Water Control & Improvement District No. 7, near Mission, Tex. Fresh concrete "struck" with rectangular blade shortly following "gun" (pl. 2, *B*). Reach just cleaned for test; straight and very smooth. This process makes a smooth surface but all rebound and loose material from the striking process must be removed completely or the canal bottom will have porous, inferior concrete. Coefficient $n=0.0122$.

No. 134, experiment FCS–97, south branch of east main canal, Hidalgo County Water Control & Improvement District No. 1, near Edinburgh, Tex. Long straight reach too far from Rio Grande for heavy muds. Surface well broomed behind "gun." Cleaned a month before test. Surface hard and sound but too rough to allow the hand to slide freely over it. Designed for $n=0.014$. One inch thick, reinforced 4- by 8-inch wire, 12-gage. Mix 1 to 4½. Cost 13 cents per square foot. Coefficient $n=0.0137$.

No. 135, experiment FCS-90, main canal, Hidalgo County Water Control & Improvement District No. 5, near Progresso, Tex. New canal of concrete, well broomed behind "gun." About 0.3 foot mud in circular bottom of 4.5 feet radius for 102° arc. No slimes on sides. Surface undulating up to 0.1 foot variation. Coefficient $n=0.0144$.

Nos. 136, 137, and 138, experiment FCS-99, main ditch, Irvine ranch, Tustin, Calif. In semicircular section, lined with concrete and left as shot, that is, there was no smoothing treatment. The contract price of 6 cents per square foot did not permit the contractor to do any polishing. The three reaches computed vary materially in surface characteristics. The section is very rough to the touch but was clean of silt and debris. It conveys clear water from a small reservoir a short distance upstream. Values of n ranged from 0.0149 to 0.0173 with a weighted average for the entire reach of 0.0165.

No. 139, experiment FCS-96, canal, Hidalgo and Cameron Counties Water Control & Improvement District No. 9, Texas. Circular section of rough concrete, probably broomed after "shooting." Slightly slimed over with silt. Coefficient $n=0.0158$.

No. 167, experiment FCS-93, lateral N. Hidalgo County Water Control & Improvement District No. 1, near McAllen, Tex. Small lateral with concrete as shot from "gun," without treatment. Very rough and clean, in section approximating parabola. Some silt slime would improve capacity. Coefficient $n=0.0176$. See FCS-92 for improvement effected by brooming.

No. 168, experiment FCS-92, same; just downstream from No. 167, the only difference being that this surface was broomed behind the "gun," making coefficient $n=0.0149$.

No. 169, experiment FCS-89½ lower canal, Lindsay-Strathmore irrigation district, Calif. Concrete as shot from the "gun" without smoothing treatment. The surface of the concrete was covered with fine algae, like velvet, without streamers. Had been recently cleaned. Water is clear and cold. The bottom, 75 percent covered with drifting dunes from 0 to 0.2 foot deep, of fine sand. Concrete both rough and undulating. Coefficient $n=0.0177$ verifies recommendation of 0.017 for clean shot concrete, without treatment. (Pl. 8, *A*.)

No. 170, experiment FCS-95, west canal, Cameron County Water Improvement District No. 1, near Harlingen, Tex. Concrete broomed behind the "gun." Silt in canal bottom about 0.1 foot thick. Wind upstream. Coefficient $n=0.0187$.

EARTH CHANNELS

No. 171, experiment CCW-14, Interstate canal, Nebraska. In same soil and with same general description as No. 191. This canal designed with frictional factor of 0.025, but on account of high velocity maximum discharge allowed is 830 second-feet instead of the 1,421 second-feet for which designed. Coefficient $n=0.012$. This value of n is almost unbelievably low, but values below 0.009 for the Sidhnai Canal in India have been vouched for by able authority (*35, p. 343*).

No. 190, experiment FCS-2, Farmers' Tristate canal, Nebraska. This test and also No. 191, are on long, straight reaches of a large canal, constructed in Brule clay. In the original design the value of n was estimated as 0.025, but, although the canal is running to but partial capacity, the mean velocity is almost sufficient to scour the material. It had one riffle midway of its length, caused by old bridge approaches jutting into the canal. The values of n in Nos. 171, 190, and 191 are comparable, as the Interstate canal is in the neighborhood of the Tristate. A fringe of grass extends along the edge but retards only a very small part of the flow. The bottom is extremely even, smooth, and hard, and with the addition of a coating of sediment from the murky waters of the North Platte River, appears to be very efficient. Gentle curves adjoin both upper and lower ends of the reach. For further notes see No. 191. Coefficient $n=0.0130$.

No. 191, experiment FCS-1, Farmers' Tristate canal, Nebraska. This reach (pl. 16, *B*) was perfectly clean cut throughout its length, and in the opinion of the author gives a better value of n than the reach in No. 190. In both tests the fall is slight, and the mean value of the results of several tests with the level was accepted. The reach is a tangent between two gentle curves. The bottom was as described in No. 190. Coefficient $n=0.0164$.

No. 192, experiment SF-15, Corinne branch, Bear River canal, Utah. Reach free of growth. Originally trapezoidal in clayey loam, but now segment of ellipse lined with smooth silt. Six years old. Coefficient $n=0.0155$.

No. 193, experiment VMC, Fort Lyons canal, Colorado. Carrying but 7 percent of capacity, merely covering bottom. Slight curves at ends of reach. Bottom is fine silt, merging into sand. In places boggy. Exceptionally smooth, regular, and free of impediments. Coefficient $n=0.0165$.

No. 194, experiment FCS-83, Maricopa canal, Salt River project. Part of a long stretch of canal with a clean, sandy bottom and a slight fringe of grass along the edge, but the influence of the latter was practically negligible (pl. 16, *C*). Coefficient $n=0.0166$.

No. 195, experiment FCS-3, Winter Creek ditch, Nebraska. A long, straight reach, with very clean, hard bottom, in a cemented material. A fringe of grass bordered both edges. A stiff wind was blowing directly downstream during the test, and a value of n of 0.0180 is probably better for this canal than that found by the measurement made. Coefficient $n=0.0170$.

Nos. 208 and 209, experiment CCW-1, Empire intake canal, Colorado. Straight, grade uniform, channel in firm sand, and gravel having no pebbles larger than one-half inch in diameter. No vegetation. Two years old. Coefficients $n=0.0170$ and 0.0194.

No. 210, experiment STH-7, Billings Land & Irrigation Co.'s canal, Montana. A straight reach of canal in clay loam soil. The little grass at the edges is of slight consequence, as the bottom is slick, though roughed by cutting in places. The mean velocity, 2.45 feet per second, was about the limit, as cutting was taking place where the bottom was not protected with a deposit of gravel. A downstream wind probably reduces the value of n from about 0.018, making it quite comparable with No. 195 above. Coefficient $n=0.0174$.

No. 211, experiment VMC, Jarbeau Power canal near Rifle, Colo. New. First third of reach in clayey loam with few water-worn stones projecting. Rest of reach in clayey loam in which moss was starting. Coefficient $n=0.0176$.

No. 212, experiment STH-5b, Cove ditch, Montana. This is the same ditch, with same channel conditions as No. 213. This reach is all curve, the first 300 feet 30°, then 300 feet on a reverse 30°. Coefficient $n=0.0180$.

No. 213, experiment STH-5a, Cove ditch, Montana. This reach is half on tangent and half on a 20° curve. Ditch 6 years old. Originally excavated in sandy loam soil, the bottom is now covered with a silt deposit. A fringe of grass retards the velocity at the edge, but not the main flow. Coefficient $n=0.0186$.

No. 214, experiment STH-19, Billings Land & Irrigation Co.'s main canal, Montana. This reach follows a gentle hillside contour, although practically straight. A little sand and fine gravel is scattered over a general bottom of clean soil. Velocity (mean 2.30 feet per second) appears to be about right for this soil, as the middle of the section is clean without cutting and there is a slight deposit of silt and mud along the sides. A downstream wind perhaps gives a value of n slightly below what might be expected. Coefficient $n=0.0181$.

No. 215, experiment FCS-82, Grand canal, Salt River project, Arizona. This reach covers a clean-cut stretch, straight except for a gentle curve about 250 feet long, shown in plate 17, *A*. Originally excavated in a clay loam soil, the section now has a deposit of clean sand in the middle and slick, silty mud near the sides. The fringe of grass shown in the view is slightly above the general high water mark and had little influence on the reach when tested. Coefficient $n=0.0183$.

No. 216, experiment SF-3, Logan, Hyde Park & Smithfield canal near Logan, Utah. In operation 15 years. Bottom and sides smooth earth and gravel up to 1 inch in diameter with some 2-inch pebbles. Slight growth of grass on one side. Value of n is lower than might be expected. Coefficient $n=0.0184$.

Nos. 217, 218, 219, and 220, experiments STH-18, Billings Land & Irrigation Co., Montana. These tests were made on the same reach of canal, with varying discharges of water. The reach is straight, with a curve nearly adjoining each end. The bottom of the canal, originally excavated in Benton shale, is covered with fine sand. The shale at the sides has broken to a fine, slick clay. The cross section is quite regular. Value of n does not vary materially. Cross winds, blowing during test *c* and *d*, might easily have affected the slope sufficiently to account for such variation as appears.

No. 221, experiment STH-6, Billings Land & Irrigation Co., Montana. Test was on a straight reach, between gentle curves. The canal, excavated in Billings clay, is generally clean, but has a little fine gravel in the bed and some fine silt deposit near the sides. A few cattle tracks and a little grass had a slight retarding effect near the sides, but did not affect the main flow. Coefficient $n=0.0188$.

No. 236, experiment FCS-87, Maxwell ditch, Colorado. This ditch follows a mountain contour. The sides were rather irregular, with a fringe of grass; the bottom was free of growth and covered with sand and fragments of rock, while the low velocity allowed a silt deposit near the banks. Coefficient $n=0.0192$.

No. 237, experiment STH-33, Bitter Root Valley Irrigation Co., Montana. This reach of canal follows a contour, giving gentle curves joined by short tangents. The bottom is covered with sand and fine gravel with an occasional cobble of two-fist size. The reach is uniform in cross section. Coefficient $n=0.0196$.

No. 238, experiment VMC, Grand Valley canal, Colorado. Carrying nearly full capacity. Bed lined with fine sediment. Sides rather uneven surface of clay loam. Short grass on bank was submerged one-half foot. Coefficient $n=0.0200$.

No. 239, experiment FCS-57, lateral 7, Turlock irrigation district, California. This canal was tested so late in the season that it was carrying but a small portion of its capacity. The reach is straight, in hard-packed smooth sand. Water being low, the grasses on the banks did not affect the flow at the time of test (pl. 17, *B*). Coefficient $n=0.0202$.

No. 240, experiment STH-17, Billings Land & Irrigation Co., Montana. This reach, located 400 feet below a tunnel and 200 feet above a flume, is in sidehill excavation of mixed earth and sand-rock with some shale. It is fairly clean, with some loose rock and sand deposits, while there is a slight growth of trailing moss at the lower end. Coefficient $n=0.0204$.

No. 241, experiment CCW-3, Rist & Goss ditch, Colorado. Built in heavy loam. Sides and bottom well coated. No weeds or aquatic growth. Coefficient $n=0.0204$.

No. 242, experiment VMC, Wilcox canal, Rifle, Colo. Bed of fine silt, sand, and pebbles, with thin scattering of 6-inch cobbles. Discharge tested less than one-twentieth of rated capacity. Coefficient $n=0.0205$.

No. 243, experiment CCW-4, Old Barnes ditch, Colorado. In good condition. Constructed in firm earth. Channel well coated with sediment. No stones or pebbles, but some long grass overhangs banks. Coefficient $n=0.0206$.

No. 244, experiment STH-34, Bitter Root Valley Irrigation Co., Montana. A canal in its fifth year of operation. Reach is straight, with bottom and sides covered with graded sand and gravel to cobble size. Sand filling the interstices between larger pieces probably accounts for a value of n far below that of a cobble ditch. Coefficient $n=0.0208$.

No. 245, experiment SF-6, Logan & Richmond canal, Utah. Bed smooth and free of vegetation. Some indentations near top of channel. Coefficient $n=0.0211$.

No. 246, experiment STH-35, Bitter Root Valley Irrigation Co.'s canal, Montana. This reach, rather irregular in form, was excavated in hardpan. Drifting sand has smoothed over some of the irregularities. Alignment is sinuous. Coefficient $n=0.0211$.

No. 247, experiment STH-14, lateral No. 2, Billings Land & Irrigation Co., Montana. A straight reach of canal, originally excavated in sandy loam soil with some gravel. Present bottom is smooth, unshifting sand, evenly distributed. Coefficient $n=0.0212$.

No. 248, experiment WBG-1, Morris canal, Louisiana. A straight section of a large rice-irrigation canal. The previous winter it had been plowed, leaving the bed rough. Water grasses retarded the velocity near the edges. The value of n is lower than the author would expect from the description. Coefficient $n=0.0216$.

No. 249, experiment STH-25, Hedge canal, Montana. A reach of canal excavated in soft granite sidehill. The present section is covered with disintegrated granite, mostly less than ½-inch size, but there are a few pieces ranging up to two-fist size. Coefficient $n=0.0216$.

No. 250, experiment FCS-78, Birch canal, Imperial irrigation district, California. Originally excavated in alluvial silt soil, but deposits of sand on the bottom and growths of grass looking like half-grown oats have completely changed the nature of the channel. The water in this valley, from the Colorado River, was heavily charged with silt at all times of the year, and this formed a slick deposit which withstands a high velocity before scouring. The conditions and values in this test and No. 315 are directly comparable, the higher value of n in No. 315 being due to the denser growth of grass as shown in the view, plate 17, *C*. Coefficient $n=0.0217$.

No. 251, experiment WBG-6, Crowley canal, Louisiana. A straight reach of rice canal. Before the beginning of the irrigation season the canal bed had been plowed and harrowed. Grass interfered with velocity near the sides. Coefficient $n=0.0219$.

No. 252, experiment VMC, Bessemer canal, Pueblo, Colo. Bed of smooth water worn adobe. Coefficient $n=0.0219$.

No. 253, experiment VMC. Same canal as No. 252. Channel the same except for the presence of cottonwood tree rootlets at the sides. Coefficient $n=0.0281$.

No. 254, experiment STH-8, high line of Big ditch, Montana. A good example of expectation for a ditch of this type. Originally constructed in a gravel soil, the low velocity has permitted deposit of silt until the bed is smoothed over and n is much smaller than it was in the new ditch. This reach follows contours with sharp curves, joined by short tangents. Coefficient $n=0.0220$.

No. 255, experiment CCW-2, Louden ditch, Colorado. Bed has clean, sandy bottom without growth of any kind. In fair condition. Coefficient $n=0.0220$.

No. 256, experiment VMC, mesa lateral, Grand Valley canal, Colorado. Bed smoothly lined with fine sediment. Sides of uneven loam. Short grass on bank submerged one-half foot. Coefficient $n=0.0220$.

No. 257, experiment FCS-64, Santa Ana and Orange canal, California. This test shows the value of cleaning a ditch to increase the capacity. The alignment (pl. 11, *A*) follows a gently curving contour. Had been well shoveled out within a few days, removing all retarding influence of grasses and moss. There was a very little soft sand near the sides of the section with occasional pockets of sand. The value of n is comparable with that in No. 307, which is on the same kind of a canal subject to the same conditions but not cleaned recently. Coefficient $n=0.0221$.

No. 258, experiment FCS-80, central main canal, Imperial, Calif. Test was on a long reach of straight canal. Banks nearly vertical as left by cleaning of silt with a bucket dredge. The bottom is very hard and quite regular, despite this method of cleaning. The velocity was retarded for about 1 foot from each bank by a growth of tules. The silt-laden waters formed slick banks. Plate 11, *B*, shows the reach and the portable rating car in action. If the sides were freed of growth at all times (impracticable in this region) the value of n would be under 0.020. For small canals in this locality see Nos. 250 and 315. Coefficient $n=0.0221$.

No. 259, experiment STH-20, Billings Land & Irrigation Co., Montana. Canal was originally constructed in varied strata, having an earth surface underlaid with a stratum of gravel, while the bed was in Benton shale. This has now been covered in places with graded gravel. In general, the upper end of the reach had a smaller sectional area, consequently a higher velocity, and the gravel was scoured clean, while the lower end had a lower velocity and the gravel influence had been reduced by the deposit of silt. Coefficient $n=0.0221$.

No. 260, experiment FCS-84, Salt River Valley canal, Arizona. A straight reach of canal originally constructed in graded gravel underlying silty loam soil. The high velocity encountered (mean 3.12 feet per second) scoured the bed of the canal, exposing hard-packed small gravel, while near the sides a slick deposit of silt formed a surface with but little retarding action on the water. The fringe of grass and small roots at the extreme edges (pl. 14, *A*), influenced but a very small portion of the flow. Coefficient $n=0.0222$.

No. 261, experiment CCW-5, Geo. Rist ditch, Colorado. Originally excavated in material ranging from earth to coarse gravel with occasional cobbles up to 6-inch size. Bed lined with sediment. Banks uneven and overhung with sod. Coefficient $n=0.0224$.

No. 262, experiment STH-2, Big ditch near Billings, Mont. Canal was originally excavated in Billings loam, which tends to be clayey. The bed has a slight deposit of sand which undercuts beneath the feet in wading, showing that the mean velocity, 2.09 feet per second, was almost sufficient to cause scouring of sand deposit. Fine mud has been deposited at the sides where the velocities are low. Coefficient $n=0.0225$.

No. 263, experiment STH-38, Bitter Root Valley Irrigation Co., Montana. The first half of this reach is on tangent, the second half on a 20° curve around a gravelly point. On the tangent the bed is covered with fine sand in serrations from 1 to 2 feet longitudinally with the canal and about 6 inches deep. In the second half the sand covers the middle portion of the bed, while gravel up to cobble size forms the edges. The value of n found, 0.0226, is lower than is to be expected.

No. 264, experiment FCS-40, lateral No. 10, Orland project, Calif. Many of the conditions holding for this test are clearly shown in plate 14, *B*. The gravel, mostly under hen's-egg size, is well compacted in the bed, while a few scattered patches of moss have a retarding influence. There were about two patches, each 5 feet in diameter, in each 100 feet of length down the ditch. Coefficient $n=0.0228$.

Nos. 265 to 285, inclusive, experiments REB 1 to 21. A comprehensive test on canals for irrigation from Rio Negro, Argentina, reported by Ballester, (*4*). Twenty-one reaches of canals and laterals, all in earth excavation, were chosen for test. The discharge was measured by current meter of the European type, using the 0.2 and 0.8 depths method, following procedure outlined by Hoyt and Grover. The slope of the water surface was taken between tops of stakes driven flush with the water surface at the two ends of the reach and every 40 to 50 meters between. The locations of the stake heads were platted on full scale vertically and at 1:2,000 horizontally, and an average line was drawn through the points as indicative of the average location of the water slope. Reaches selected clearly showed either silt accumulations or freedom from such deposits. This selection was for the purpose of comparing Kennedy's silt data (*32*) with experience in Argentina. Detailed comments for the various reaches follow (the numbers correspond with Ballester's series numbers):

1 to 4, inclusive. In good condition, without silt deposits.

5. Gravel bed and vertical sides. Almost a "cobble-bottomed" canal.

6. Gaged at maximum capacity; bed of gravel, somewhat irregular; apparently with silt deposits at the feet of the side slopes, the bed rounded.

7. Straight, the trapezoidal original section held intact, with no silt beds after 4 years of operation.

8. Bed in sandy soil, with some vegetation at the sides. After 15 years of operation, no silt banks.

9. Canal in sandy soil; annual cleaning of silt banks required.

10 to 14, inclusive. Representative of canals in excellent and economic maintenance, both in respect to silt banks and to growing aquatic plants. Tests 10 and 11 show rich grass growth along the banks at the water surface.

15 and 16. Little silt deposit and abundant side vegetation which retards the flow so that cleanings are required every 2 to 3 years.

17. Silts itself so that annual cleanings are required.

18. A reach operated more than 15 years without requiring desilting.

19 and 20. Reaches in good condition, without silt banks but with lateral vegetation that retards the flow, explaining the high value of the roughness coefficient.

21. A reach with both silt and aquatic growths that reduce the velocity "extraordinarily" as indicated by the high coefficient.

No. 286, experiment SF–39, lateral 2, Bear River canal, Utah. Conditions similar to No. 287 except there was no moss and some bunches of grass along the edges retarded velocity. Coefficient $n=0.0230$.

No. 287, experiment SF–14, lateral of Bear River canal, Utah. Excavated in clayey loam. Now bed is covered with sediment. Patches of horsetail moss. Edges uneven. Coefficient $n=0.0230$.

No. 291, experiment FCS–39, main canal south, Orland project, Bureau of Reclamation, California. This reach is straight; originally constructed in a yellow clay which is very slick when wet. A darker deposit of silt now covers much of this clay. A value of n of about 0.017 might be expected but for patches of moss and water grasses occupying about 20 percent of the bottom of the channel. This influence brings about a value of 0.0231 for n.

No. 292, experiment FCS–36, River Branch canal, Sacramento Valley Irrigation Co., California. Excavated in Sacramento silty clay loam, which breaks into very hard small clods (pl. 14, *C*). The bed of the ditch was very slick and hard. A few scattered soft lumps of mud and a fringe of grass and thorns extending out 1 foot raised the value of n from about 0.017 to 0.0236. Although the mean velocity is nearly 3 feet per second, there was no sign of scour in this hard soil. Coefficient $n=0.0236$.

No. 293, experiment SF–1, Providence canal, Utah. In gravel size of peas with scattered pieces size of walnuts. No vegetation. Coefficient $n=0.0238$.

No. 294, experiment SF–27, Logan, Hyde Park and Thatcher canal, Utah. Sides smooth with sediment; bottom of earth, gravel, and pebbles up to 2½ inches diameter. Coarser material covered one-fourth of perimeter. Coefficient $n=0.0246$.

No. 295, experiment WBG–4, a small, new ditch in Louisiana. This ditch was practically as left by a plow, being but a week old. The reach was straight. Coefficient $n=0.0246$.

No. 296, experiment SF–18, College and City canal, Utah. No vegetation, but sides uneven. Bed covered with fragments of flat rock from ½ to 2 inches in greatest dimension. Coefficient $n=0.0247$.

No. 297, experiment FCS–88, Boulder & White Rock ditch, Colorado. A small ditch with one bend. Original excavation was in meadow soil over river gravel. The bed contained graded gravel, mostly small but with a few cobbles of two-fist size. A dark silt had deposited in the lower velocities near the edges which were nearly vertical, well-sodded banks. This ditch would be called in a good working condition as most of the stones were unavoidable. Coefficient $n=0.0248$.

No. 299, experiment STH–3, Billings Land & Irrigation Co., Montana. This reach was originally excavated in Billings gravel. Silt has deposited in the low velocities at the sides, but the main bed is composed of gravel with cobbles up to two-fist size. The slight fringe of grass did not retard the main flow. Coefficient $n=0.0258$.

No. 300, experiment STH–1, Billings Land & Irrigation Co., Montana. A straight reach excavated in gravelly soil. The bed is of compact gravel up to one-fist size, while silt has deposited in the lower velocities near the sides. No grass in the water section. The mean velocity of the water, 2.35 feet per second, appears to be sufficient to prevent the deposit of silt over the bed, though the water is very muddy. Coefficient $n=0.0259$.

No. 301, experiment STH–36, Bitter Root Valley Irrigation Co., Montana. The first two-thirds of the reach was on a sidehill in hardpan, while the last third, originally constructed on a creek bottom, is now formed of sand drifts similar to those spoken of in No. 263. In the first part the hardpan is scoured clean except on inside of curves where sand has deposited, while farther down the ditch some cobbles are mixed with the sand. Coefficient $n=0.0260$.

No. 302, experiment FCS–14, North Ogden canal, Utah. This reach follows a hillside contour about one-half mile below the mouth of Ogden Canyon. The material is composed of soil and rounded boulders ranging from sand to stones several hundred pounds in weight. The sides were quite vertical and fringed with willow roots and grass, while a few patches of moss were scattered throughout the length of the reach. Aside from this moss this test would come under the class of cobble-bottom ditches, and the value of $n=0.0262$ is about right for such ditches.

No. 303, experiment FCS–56, main branch canal, Turlock irrigation district, California. This canal, shown on plate 18, *B*, was carrying but a small part of its total capacity. The water was so low that the influence of grass, which would have affected a deeper section, was lost. The bed was hard-packed fine sand. Coefficient $n=0.0262$.

No. 304, experiment VMC, Rocky Ford canal, Colorado. Bed of fine loose sand. Sides of clay with fine grass roots projecting. Some grass overhangs into water. Canal somewhat crooked and banks irregular. Coefficient $n=0.0266$.

No. 305, experiment FCS–23, a lateral of the South Side Twin Falls canal, in Twin Falls, Idaho (pl. 18, *A*). Excavation was through 1 foot of lava-ash soil before striking hardpan. The present bed of the lateral is clean and hard. A dense growth of sod and long grass retards the water at the vertical sides, and some silt has deposited on the edges of the bottom in the low velocities. Coefficient $n=0.0267$.

No. 306, experiment FCS–8, Salt Lake City and Jordan canal, Utah. A reach with one gentle curve in the upper end, but otherwise straight. Originally constructed in sandy soil with small gravel, the bottom now is very hard and cemented except at the sides, where the velocity is not sufficient to prevent silt from depositing. A dense growth of grass killed the velocity for about one-half foot from the vertical banks, typical of old ditches in Colorado and Utah. Coefficient $n=0.0267$.

No. 307, experiment FCS–66, Fullerton ditch, Anaheim-Union Water Co., California (pl. 18, *C*). Grass and moss kill the velocity for about 1 foot from the banks; the bed is a hard, cemented, sandy loam, with about 0.1 foot of shifting sand. This canal was under exactly the same conditions as No. 257, except that this needed cleaning and the other had just been cleaned. Coefficient $n=0.0269$.

No. 308, experiment FCS–86, Farmers' canal, near Boulder, Colo. This canal is on a gently curving hillside. Willows and grass form a dense fringe at the sides, while the bottom, originally in red mountain soil mixed with fragments of sandstone, now has a hard, gravelly bottom, with angular fragments rather than rounded pebbles. The section is irregular, and the value of n is quite comparable with that of a cobble-bottom ditch, although this reach is not cobble bottomed. Coefficient $n=0.0270$.

Nos. 309, 310, and 311, experiments STH–12a, b, and c, lateral 2, Billings Land & Irrigation Co., Montana. An irregular section of small lateral, constructed in loamy earth. Now fringed with grass, which trails somewhat, though newly mowed. The bottom is irregular, with drifting sand throughout most of the reach. The canal is too irregular to justify conceding too much weight to the various values of n found.

No. 312, experiment FCS–5, lateral of Parley's ditch, Utah. A straight reach in gravelly loam soil. A fringe of grass retards velocity for about one-half foot from the banks. Bottom is clean sand which yields about 1 inch to the feet of a wader. A slight wind upstream makes the value of n a little high. Coefficient $n=0.0278$.

No. 313, experiment VMC, Bessemer ditch, Colorado. Bed of fine silt merging into clays with liberal sprinkling of loose stones up to 3 inches diameter. Coefficient $n=0.0280$.

No. 314, experiment FCS–20, lateral of South Side Twin Falls canal, Idaho. On a gentle contour curve of about 400 feet radius. Constructed in hardpan underlying about a foot of lava-ash soil, the water section is quite slick but is badly washed in longitudinal gullies. The banks, too, are irregular and fringed with a dense growth of grass and alfalfa. Coefficient $n=0.0283$.

No. 315, experiment FCS–79, Beech canal, Imperial Valley, Calif. This test is under exactly the same conditions as those described in No. 250, with the exception that a longer time has elapsed since the dense growth of grass shown on plate 11, *C* was cut. The difference is noted by comparing the above view with that in plate 17, *C*. Coefficient $n=0.0290$.

No. 316, experiment FCS–49, Wheeler ditch, Nevada. Ditch follows contour in curves of from 300 to 500 feet radius. While the bottom is hard many boulders of several hundred pounds weight border the channel, and a few have rolled into it. A dense growth of grass and bushes fringes the sides, and a fine, dense moss retards velocity for about 0.3 foot from the bottom. Coefficient $n=0.0292$.

No. 317, experiment FCS–85, lateral 10 from Arizona canal, Salt River project, Ariz. Straight reach of small ditch on a steep grade. Constructed in a silt loam soil, the high velocity has washed very irregular gullies and pockets. An average growth of grass and weeds also retards velocity. Coefficient $n=0.0298$.

Nos. 318 and 319, experiments STH–13, lateral of Billings Land & Irrigation Co., Montana. These tests on the same reach of canal but with varying discharges, and for No. 318 grass fringing the edges was cut, hence removing some of the retarding influence present in No. 319. The bottom is covered with fine, deep, shifting sand.

No. 320, experiment FCS–72, upper canal, Riverside Water Co., California. A straight reach originally excavated in a sandy loam soil. The bottom is covered with a bed of fine sand which remains hard until disturbed, when it cuts rapidly. Dense grass along the sides and scattered patches of moss in the canal cause a high value of n. Coefficient $n=0.0315$.

No. 321, experiment FCS–77a, lower canal, Riverside Water Co., California. Though originally excavated in a clean-cut section of soft hardpan, the present condition of this reach is much less efficient, owing to a deposit of shifting sand and growths of water grasses and dense grass along the edge, though not so bad as in No. 322. Coefficient $n=0.0318$.

No. 322, experiment FCS–77b. Same canal as No. 321, but the grass along the edge kills velocity for about 1 foot from both banks. Otherwise the same general condition holds as before. This reach adjoins the other reach at a right-angled bend. Coefficient $n=0.0360$.

No. 323, experiment FCS–59, main canal, Modesto irrigation district, Calif. Test was made on a wide canal carrying but a small portion of its total capacity. This gave a condition of shallow water flowing over gullied hardpan having about 0.1 foot of shifting sand. No grass touched the water at this stage. Coefficient $n=0.0300$.

No. 324, experiment SF–64, Hyrum canal, Utah. Sides overgrown with alfalfa and weeds. Bed of coarse gravel up to walnut size. Coefficient $n=0.0319$.

No. 325, experiment VMC, Bessemer canal, Colorado. Bed of fine silt, merging into clays with liberal sprinkling of loose stones up to 3 inches diameter. Coefficient $n=0.0321$.

No. 326, experiment FCS–9, lower canal from Big Cottonwood Creek, Utah. Originally constructed in a sandy loam soil, the bottom was hard with no cobbles but with a deposit of soft mud in the lower velocities near the sides. The reach tested is on a gentle contour curve, which exerts an inappreciable influence when compared with the vegetable growth. The banks are nearly vertical and irregular, like most rooted channels. Willow roots and grass have so encroached on the channel that a high value of n is obtained. Coefficient $n=0.0324$.

No. 327, experiment FCS–60, Yosemite Power Co.'s ditch. Follows a mountain contour in disintegrated-rock soil. A fringe of bushes and grass retards velocity at the banks, while the bottom is porous and gravelly with scattered boulders and rock fragments up to two-fist size. Coefficient $n=0.0334$.

No. 328, experiment FCS–62, Golden Rock ditch, Yosemite Power Co., California. Reach tested follows a gentle mountain contour. The bottom is of clean disintegrated slate with scattered pieces to two-fist size. Although this bottom has a great retarding influence, the value of n is higher than is to be expected. There was but little grass touching the water. Coefficient $n=0.0346$.

No. 329, experiment STH–15, lateral 1, Billings Land & Irrigation Co., Montana. This reach of ditch is in sandy loam fill. The water section is irregular with sand in the bottom and a little trailing grass and moss. Coefficient $n=0.0349$.

No. 330, experiment SF–26, Logan and Benson-Ward canal, Utah. Bed of medium-sized gravel. Flow much impeded by horsetail moss occupying about one-fourth of water section. Coefficient $n=0.0352$.

No. 331, experiment FCS–18, Logan and Hyde Park canal, Logan, Utah. Straight reach originally constructed in gravelly soil with many cobbles from egg to two-fist size. At time of test the edges were irregular and densely grassed, with patches of moss and some cobbles scattered throughout the reach. The moss lies usually within 0.3 foot of the bottom. Coefficient $n=0.0364$.

No. 332, experiment CCW–9, Hillsboro ditch, Colorado. In general bad condition, with irregular gradient and rough banks. Gravel channel scoured by current. Coefficient $n=0.0371$.

No. 333, experiment FCS–53, lateral 2½, Turlock irrigation district, Calif. Straight reach excavated in hardpan requiring blasting. This leaves the section rough and pitted. The bottom was covered with 3 inches of rough, gritty sand. As the canal was only about one-half full, no grass touched the water. Coefficient $n=0.0373$.

No. 334, experiment FCS–25, Perrault canal, Boise, Idaho. This ditch, constructed in gravelly loam soil, has a very hard cemented bottom. A dense growth of grass so kills the velocity for a distance of about 1 foot from each bank that the value of n is very much greater than would be the case if the canal were kept free of this growth. Coefficient $n=0.0381$.

No. 335, experiment, SF–62, lateral of Hyrum canal, Utah. Partially overgrown with alfalfa. Strip of moss on each side occupied about one-fifth of channel. Bed of flat fragments of rock, up to 3 inches greatest dimension. Coefficient $n=0.0393$.

No. 336, experiment FCS–46, Orr ditch, Reno, Nev. The reach in a horseshoe curve around the small lake on the university campus. Originally excavated in loamy soil with a little gravel. The bed was clean scoured, but near the sides a man wading sank about 4 inches in soft mud. Dense grass and willows retard the velocity at both banks, while the water section is very irregular throughout the reach. Coefficient $n=0.0397$.

No. 337, experiment FCS–21, a small ditch in Twin Falls, Idaho. Originally constructed in hardpan, the reach is irregular with scattered debris such as is so often found in town ditches. Grass arches across in many places. Coefficient $n=0.0399$.

No. 338, experiment FCS–4, New Rutner ditch, Nebraska. Ditch follows a gentle contour line down a creek bottom. At time of test it had a very hard bottom of medium-fine gravel, well packed, but a dense growth of grass killed the velocity for about one-half foot from each bank, and scattered patches of moss retarded that in the middle. The banks of the ditch are very irregular. Coefficient $n=0.0436$.

No. 339, experiment FCS–44, Sullivan & Kelly ditch, Nevada. A ditch excavated in a gravel and cobble hillside. At time of test the bottom was hard-packed gravel in the center with a slight deposit of soft mud at the sides. Scattered cobbles and a dense growth of grass retarded the velocity of the water. The reach follows a gently curving contour line with one right-angled bend near the lower end. Coefficient $n=0.0436$.

No. 340, experiment WBG–2, Roller canal, Louisiana. This test was made on a straight reach of canal. The vegetation extended about 5 feet from each shore. Coefficient $n=0.0461$.

No. 343, experiment SF–53, lateral of Thatcher canal, Utah. About two-thirds filled with horsetail moss. Such bed as is exposed is sediment. Coefficient $n=0.0519$.

No. 344, experiment SF–52, lateral of Thatcher canal, Utah. About three-fourths filled with moss. Bed exposed is fine sediment. Coefficient $n=0.0529$.

No. 345, experiment WBG–5, a small ditch in Louisiana, chosen as representative of the small ditches in the rice country. Grass extended from one bank to the other across the bed, occasionally growing to the water surface from the bottom of the ditch. The grass forms a dense mat in the bottom. It had been cut with a scythe about 1 week before the test. Coefficient $n=0.0544$.

COBBLE-BOTTOM DITCHES

No. 346, experiment CCW–13, Beasley ditch, Boulder, Colo. Channel of gravel on bottom with cobbles on the sides. On a slight curve with no vegetation in channel. General condition classed as good. Coefficient $n=0.0220$.

No. 347, experiment VMC, Rio Grande canal, lateral 1–c, Del Norte, Colo. Description about the same as for No. 348, except that the gravel is finer. Coefficient $n=0.0221$.

No. 348, experiment VMC, Rio Grande canal, Del Norte, Colo. Bed varies from fine gravel to smooth round rocks about 6 inches in diameter. Coefficient $n=0.0284$. (Pl. 22, *C*.)

Nos. 349 to 370, inclusive, experiments BCC, Cachapoal canal, Braden Copper Co., Chile. Originally constructed in 1909–11 for capacity of 550 second-feet. Enlarged in 1928 to 750 second-feet. Prior to this enlargement some 480 cross sections were taken; capacity flows were held steady and measured at both ends of canal. Reaches are listed in order, with distance in kilometers from head, in column 2, table 1. Elevation of water surface at each measured section was used in determination of *C* in Chezy formula for typical reaches listed covering length of canal of 12 kilometers. Typical condition indicated by plate 21, *B*.

No. 375, experiment SF–63, Hyrum canal, Utah. Sides of earth. One-half of perimeter across bottom covered with rock fragments up to 1 inch across. Weeds and alfalfa grew up to water's edge. Coefficient $n=0.0260$.

No. 376, experiment STH–31, Bitter Root Valley Irrigation Co.'s canal, Montana. A nearly straight reach excavated in very gravelly ground with boulders up to 2 cubic feet in size. The first third of the distance is fairly smooth on the bottom with no stones larger than two-fist size; upper slope cobbly and with some grass on the lower water edge. Second third of distance shows much roughness, with cobbles over most of bottom. Last third, in condition intermediate between first and second thirds. Coefficient $n=0.0262$.

No. 377, experiment STH–4, Billings Land & Irrigation Co., Montana. A straight reach excavated in graded gravel up to two-fist size underlying about 1 foot of soil. The canal is 9 years old. Mud has deposited in the slower velocities at the sides. Dense grass fringes the edge but does not trail in the water. Coefficient $n=0.0264$.

No. 378, experiment CCW–7, Loveland and Greeley canal, Colorado. Overhanging sod banks with grass in water. Earth channel with many cobbles up to 8 inches in diameter. Reach on a curve. Coefficient $n=0.0267$.

No. 379, experiment FCS–10, upper canal from Big Cottonwood Creek, Utah. Reach follows a gentle contour curve on a very gravelly hillside. Sides vertical and lined with trees and willows, rootlets of which extend into the water prism. The bottom is completely covered with cobbles up to two-fist size. As with nearly all ditches of this character the sides are irregular in outline, the cobbles not breaking into an even bank. Coefficient $n=0.0277$.

No. 380, experiment FCS–15, Logan and Northern canal, Utah. A fine example of ditches following gravelly hillside contours near the mouths of canyons, a condition typical of many canals near the mountains. The sides are densely fringed with willows and bushes, rootlets of which hold silt and form nearly vertical banks. The bottom is completely covered with well-packed gravel and cobbles up to two-fist size. Coefficient $n=0.0270$.

No. 381, experiment VMC, Rio Grande lateral No. 1, Colorado. Bed of graded material from fine gravel to water-worn rocks 6 inches or more in diameter. Three tests on the same reach with 380 second-feet, 33.34 second-feet, and 27.16 second-feet, gave values of n as 0.0284, 0.0386, and 0.0370, respectively. This gives a lower value of n for the greater discharge.

No. 382, experiment FCS–51, Reno ditch of the Reno Light & Power Co., Nevada. Reach was originally excavated in an old river bed containing innumerable boulders up to 5 cubic feet in size (pl. 20, A). About a year before this test the sides and bottom of this channel had been paved with a hand-laid riprap of these boulders, the sides being laid about ⅓ to 1, and the bottom flat. Many of the boulders at the top of the walls have rolled into the canal, as no cement or other bond was used, but the general condition of the walls appears to be good. The reach tested was straight with the exception of a gentle curve in the last 200 feet. Coefficient $n=0.0291$.

No. 383, experiment CCW–12, Beasley ditch, Boulder, Colo. Excavated in gravel and fine sand with a good many small cobbles. Banks held in place by logs laid parallel to stream. Coefficient $n=0.0320$.

No. 384, experiment FCS–43, Sullivan & Kelley ditch, Nevada. This reach follows a gentle contour on a rocky hillside. The boulders have been hand laid

in a nearly vertical wall on the lower side while the bottom and upper side are rough and irregular with projecting large boulders. A slime of mud from the Truckee River water covers all rocks below the water line. Vegetation is negligible. Coefficient $n=0.0324$.

No. 385, experiment SF–55, Smithfield canal lateral, Utah. Cobbles partially covered with silt. Edges made irregular by cattle. Coefficient $n=0.0329$.

No. 386, experiment SF–47, Logan and Hyde Park Canal, Utah. Entire channel composed of loose coarse gravel up to hen's egg in size. Coefficient $n=0.0337$.

No. 387, experiment SF–33, lateral of Hyrum canal, Utah. A case where water flows very slowly over a rough surface on steep grade. Bed composed of loose cobbles up to 3-inch size. Coefficient $n=0.0365$.

No. 388, experiment SF–57, lateral of Smithfield canal, Utah. Canal on steep grade with bed composed of gravel and cobbles. Coefficient $n=0.0377$.

No. 389, experiment FCS–42, Cochrane ditch, Nevada. Constructed in gravelly soil with many cobbles. Test was made on a reach having one bend. The bottom had many loose cobbles scattered on an otherwise hard gravel bed. The banks throughout nearly all of the reach tested were overhung with densely grassed sod. In addition about 20 percent of the water section was occupied with moss. Coefficient $n=0.0379$.

No. 390, experiment FCS–89, Beasley ditch, Colorado. A straight reach carrying but about one-fourth its capacity. The bottom and sides are a mass of unpacked gravel and boulders up to 2 cubic feet in size. Coefficient $n=0.0383$.

No. 391, experiment FCS–52, Capurro ditch near Reno, Nev. A small ditch thickly fringed with grass and with scattered cobbles in the bottom. The banks are nearly vertical, densely rooted, and very irregular. The bottom is covered with about 0.1 foot of soft mud through which the scattered cobbles project. Coefficient $n=0.0403$.

No. 392, experiment SF–24, canal of Brigham City Electric Light Co., Utah. Well-formed canal. Bed of medium-sized unpacked gravel. One-third of water section filled with long waving water plants. Coefficient $n=0.0424$.

No. 393, experiment SF–56, lateral of Smithfield City canal, Utah. Edges uneven. Bed of clean-washed gravel and cobbles. Coefficient $n=0.0423$.

No. 394, experiment SF–46, Brigham City canal, Utah. About half full of horsetail moss. Such bed as is exposed is gravel. Edges overgrown with cress and weeds. Coefficient $n=0.0499$.

SIDEHILL CUTS, WITH RETAINING WALLS

No. 395, experiment STH–23, Hedge canal, Montana. A short reach excavated in granite hillside, with the lower bank formed of a random rubble masonry wall, well plastered with mortar on the nearly vertical water face. The reach is straight except for a slight curve in the last 50 feet. The bottom is covered with granite gravel, most of which would pass a ½-inch screen, with occasional pieces one-fist size. The excavation is quite true to line for a rock cut. Coefficient $n=0.0185$.

No. 396, experiment STH–27, Hedge canal, Montana. This reach is about 500 feet below that in No. 395. The upper side is excavated quite true to line in earth and disintegrated granite. The lower side is a vertical concrete wall laid against board forms. The floor is concrete with from 1 to 2 inches of fine, sharp ravelings from the hillside. In spots the floor shows. Coefficient $n=0.0225$.

No. 397, experiment STH–9, Cove ditch, Montana. Reach is cut from a sandstone hillside. Lower bank is a rubble masonry wall plastered with concrete on the water side. Bottom is also overlaid with concrete. Alinement is wavy with some 20° curves. From stations 0 to 2 there is some gravel; from stations 2 to 4, clean bottom, the rock on upper bank being smooth but the width irregular; from stations 4 to 6, more uniform in width but rough on rock side; from stations 6 to 7, rock wall rough and width variable; from stations 7 to 8, rock wall smooth, bottom clean or little gravel, the width uniform. Coefficient $n=0.0228$.

No. 398, experiment FCS–17a, Logan, Hyde Park, and Smithfield canal, Utah. A reach between an earth section and the reach in No. 399 below (pl. 15, *A*). The excavation is on a steep hillside. The upper bank is mostly of willow roots, while the lower bank is a well-made concrete wall. The bottom is covered with coarse gravel. This reach is nearly straight with bends at both ends. Coefficient $n=0.0256$.

No. 399, experiment FCS–17b, Logan, Hyde Park, and Smithfield canal, Utah. Just below the reach described in No. 398 the canal enters the section covered in this test. The same concrete wall formed the lower bank, and the bottom was about the same, but the upper bank was a rough vertical rock cut. The difference in the value of n is about what is to be expected. Coefficient $n=0.0278$.

No. 400, experiment STH–26, Hedge canal, Montana. A rock cut with concrete floor and a rubble masonry lower wall, faced with 3 inches of concrete deposited against wood forms. The bottom is mostly covered with sand and ravelings of small rock. The upper bank is rough rock excavated true to cross section. The alinement is practically straight except for one sharp curve. Coefficient $n=0.0269$.

MISCELLANEOUS SECTIONS

Nos. 404 and 405 experiments FCS–74 and FCS–73, lower canal, Riverside Water Co., California. These tests made on a straight reach of canal in a sandy soil with a shifting sand bottom and a wood lining on the lower side (pl. 15, *B*). The canal in test No. 404 is in the shade of a dense row of trees and is free of moss accumulations. Coefficient $n=0.0249$. The canal in test No. 405 is in the sun and moss has accumulated on the wood lining. Coefficient $n=0.0291$.

In both tests the water was retarded by a rank growth of grass for about 1 foot from the bank opposite the wood lining. The difference in the values of n is directly due to the moss, which grows in sunlight but not in shade.

Nos. 406 and 407, experiments BR–S–26 and 27. Rossi Mill ditch, Idaho. Rough board sides with gravel. Some grass growing through cracks between boards. Coefficient $n=0.020$.

No. 408, experiment FCS–17c, Logan, Hyde Park, and Smithfield canal, Utah. This reach is fairly straight, excavated in rough rock. The bottom is strewn with coarse gravel (pl. 15, *C*). Coefficient $n=0.0298$.

No. 409, experiment JE. Canal for electric plant of city of Aarau, Switzerland (*62*). A straight canal with large geschiebe on the natural bed and concrete sides on slope of 1 to 1. Coefficient $n=0.0173$.

No. 426, experiment FCS–AK, Drum canal, Pacific Gas & Electric Co., California. A test by representatives of the owners and the author. The canal section lies on a steep mountain side (pl. 19, *A*). The upper bank is a submerged rock wall 4 feet high. Above this the canal has been widened abruptly, forming a berm 5 feet wide, to an excavated side slope. The water was 3 feet deep on the berm. The lower side is paved with concrete "planks" laid side by side down the incline of the bank. These planks are precast and used wherever erosion endangers a canal bank. Cross sections had been carefully taken at every station through the reach tested, with the water out of the canal. The sectional areas varied greatly over a range about 13 percent above and below the computed mean area. Coefficient $n=0.0262$.[15]

No. 428, experiment FCS–100, Deschutes municipal district, near Bend, Oreg. A straight reach of canal with lava-rock masonry walls having plastered inside face, the bottom of fairly smooth concrete (pl. 19, *C*). Cross sections were taken every station through the length tested. Coefficient $n=0.0207$.

No. 429, experiment FCS–98, Yakima Valley canal, near Yakima, Wash. A combination of concrete lining and bench flume, which replaced a timber flume after the latter wore out. The upper side is on a slope of about ½ to 1. The bottom is a typical concrete lining with some rock debris. The lower side is a nearly vertical concrete wall like a bench flume. The meter measurement was made at the company gaging station just below the outlet of Cowiche siphon. This point was the upper end of the reach tested. The value of $n=0.0154$ is typical of concrete linings subject to hillside debris.

No. 430, experiment BR, Cottonwood flume, Idaho. Rubblestone fairly well laid. Some coarse sand in bed being carried as silt. Coefficient $n=0.0163$.

No. 431, experiment FCS–29, Jacobs ditch, Boise, Idaho. As shown in plate 9, *B* the sides are of first-class rubble masonry with all cracks smoothly plastered with cement. The bottom is smooth cement lining laid like a good grade of sidewalk. About 50 feet below the lower end of the reach the ditch passed through a vertical trash grating, which was kept clean of debris during the test. Coefficient $n=0.0149$.

No. 432, experiment FCS–27. The same ditch as No. 431, but this reach is lined on both sides and bottom with dry laid, unchinked rubble, as shown in plate 9, *C*. The bottom is quite irregular, with scattered loose cobbles. A clean grating came a short distance below the reach, as in No. 431. Coefficient $n=0.0235$.

[15] KIDDER, A. W. REPORT ON TEST FOR COEFFICIENT OF ROUGHNESS AND RETARDATION OF FLOW DUE TO CURVATURE DRUM AND LAKE VALLEY CROSS-OVER CANALS. Drum Division Pacific Gas & Electric Co. 6 p. illus. (Unpublished.)

No. 433, experiment FCS–28. The same ditch as No. 431. This reach, one city square long, came between No. 431 and No. 432. It appeared to have been originally like No. 431, except that the bottom was not lined. There were several cobbles throughout the bottom of this reach, and this probably accounts for the fact that the value of n is slightly higher than for No. 432, while to all appearances it should have been slightly lower. A grating similar to those noted above came below the lower end of the reach. Coefficient $n=0.0250$.

No. 434, experiment FCS–45, Orr ditch, Reno, Nev. As shown in plate 16, *A*, the sides are smoothly built rubble, with most of the cracks well plastered, but the bottom is covered with shifting sand and loose cobbles so that, if lined, the lining is completely concealed. The lined section ends shortly below the lower end of this reach, passing into an earth channel. This test exemplifies the need of keeping sand and gravel from a lined section if the low value of n that might be expected is to be realized. Coefficient $n=0.0298$.

CONCRETE CHUTES

Nos. 451 and 452, experiments VMC, south canal, Uncompahgre project of Bureau of Reclamation, Colorado. This canal, extending from the outlet of Gunnison tunnel to the Uncompahgre River, is a series of ordinary concrete-lined canal sections (see Nos. 63 to 67, inclusive) with several steep chutes (pls. 10, *A* and 22, *B*). Tests were made on the latter by Cone and his associates some 3 years before the Foster test (No. 453) and 19 years before the more elaborate tests by Lane and associates (Nos. 454 to 464, inclusive).[16] The Foster tests were reported in Bureau of Reclamation data card No. 25. The photographs show the class of concrete when the canal was new and detail views show how the bottom and lower parts of the sides have been eroded by the high velocities (20 to 35 feet per second) in a terrain that contributes much abrasive material.

PRACTICAL USES OF THE EXPERIMENTAL DATA

The greatest practical use of the data given in table 1 is to allow selections of values of Kutter's n for use in the design of new irrigation and similar channels for specified original surfaces of definite categories with the indicated modifications of these values that may reasonably be anticipated to result from seasonal or permanent changes. Likewise to be anticipated are the changes in the values of n reflecting conditions within the water prism that affect the hydraulic roughness but may have no bearing whatever on the condition of the surface of the conduit material. Obviously, canals should seldom be designed so that the best possible conditions are anticipated, for if they are not attained the channel fails to meet capacity requirements.

The use next in importance is that made by the management of any system involving older canals. Such managements are continually faced with the problem or changing from simple earth channels to a more improved type. This may be necessary to stop seepage and water-logging of land adjacent to canals, to increase the capacity of the canal without increasing its size, or to hold the same capacity in a smaller channel. In some highly developed areas, canals located beside roads have been placed in covered conduits in order to make more space available for highway traffic. For instance, the Sanderfer Ditch near Whittier, Calif., (pl. 5, *C*) has been placed in a concrete pipe operating as a flow-line channel i. e., not under pressure.[17]

Similar changes have been made in orange groves and other farm land where the value of the areas salvaged and reduction of operating costs have more than paid for the exchange of covered concrete con-

[16] LANE, E. W. AN INVESTIGATION OF THE HYDRAULICS OF A LONG CHUTE IN THE SOUTH CANAL OF THE UNCOMPAHGRE PROJECT, U. S. Bureau of Reclamation, 12 p. illus. (Unpublished.)

[17] A change like this involves the capacity of concrete-pipe lines. For this information the reader is referred to Department Bulletin 852 (*56*). For sale by the Superintendent of Documents, Washington, D. C., at 25 cents.

duits for original open canals in earth or concrete. Another use is described in detail on page 15 where the capacity of an existing conduit was increased by direct improvement of the original channel.

A future use lies in the studies of the canalization of streams. Much of this work has been done in Europe. In the United States many small streams, in their passage through urban property, have been placed in canal sections, either open or covered. Recently the Rio Grande has been rectified from El Paso to Fort Quitman, Tex. Funished, summer of 1938.) This work has changed a meandering river for 155 miles of its length to a canal but 88 miles in length, with the resulting steeper gradient available for the transportation of silt and sand. This change also stabilized the international boundary between the United States and Mexico. The canalized channel was designed for a flow of 11,000 second-feet, using a value of Kutter's n of 0.025 for the normal flow channel and 0.030 for the flood channel.

As problems arise, the usual given quantities are: The discharge, Q, and generally the bed slope s. Uniform flow must ordinarily be assumed; hence s is parallel and equal in value to the energy slope S. Sometimes any slope may be used, over an appreciable range, so that many possible channels can be set up for economic study. Even with both Q and S given, there are many shapes and sizes that will yield the desired answers from a mathematical standpoint. Most canals will have gentle velocities without entrained air so that the continuity equation, $Q=AV$, will hold true. Thus any combination of A and V that will yield the desired Q can be used. But solution of the Kutter or the Manning formula yields a value of R rather than of A. For preliminary study a set of tentative simultaneous values of R and V can be taken from the estimate diagram (p. 66). Then $A=\frac{Q}{V}$. Reference to the Bureau of Reclamation hydraulic tables (*67*) readily shows several combinations of canal dimensions and shapes, all yielding the required values of R and A.

The data herein can be used for a question frequently propounded to the engineer, What will be the maximum capacity of a canal, spillway, floodway or the like, already constructed but never subjected to full capacity? Here the slope S is generally assumed as equal to the bed slope s, unless other facts are available from which backwater curves can be developed or other elements that give a more proper value of S, even if nonuniform flow will be found to hold.

From the measurements of cross section, values of A, P, and R can be developed for all depths. With the best possible estimate of the value of n that will hold at the time given in the hypothetical question, then the estimate diagram can be entered with the various possible combinations of R, S, and the assumed n. The diagram will yield the corresponding values of V which can be multiplied by the proper value of A to produce the desired answer, Q. The maximum Q of the various possibilities can thus be estimated. Of course, all such estimates are approximations only.

Before reaching decisions in such circumstances, the engineer should consider:

1. The material of construction and the channel shapes available for it.
2. The most efficient proportions for various shapes.

3. Whether the canal will be of moderate slopes and in sinuous flow or on a steep slope and thus in shooting flow.
4. Whether the terrain permits of a relatively straight canal or calls for many curves and bends.
5. The surface of the material. A good surface in any material does not tell the whole story. Other things must be considered because hydraulic friction frequently differs from surface friction.
6. Whether the capacity will be affected by muddy water, insect life, or aquatic growths.
7. With items 1 to 6 considered, what velocities should be used.

These items are discussed in detail from the capacity and design standpoint in the following pages.

MATERIAL AND SHAPE OF CHANNEL

Usually the various materials of which a canal can be constructed are associated with certain definite shapes.

Earth.—Original construction in a trapezoid with some assurance that this shape will tend to become an elliptical bed (pl. 13, *A*). The form of the ellipse depends largely on the amount of gravel and cobbles in the matrix (*35, p. 273*). Fine silts without grit tend to produce a deep narrow section. Wide, flat bottoms are often developed where cobbles are predominant (pl. 22, *C*). Intermediate shapes appear to depend on the relative number and size of pebbles and boulders.

Concrete.—Generally a trapezoid with side slope of 1½ or 2 to 1. In lower Rio Grande Valley common shapes with shot concrete are 120° of a circle with tangent sides or an approximate parabola. These shapes have enough arch action to resist breaking under pressure of earth or mud backing. Many modern canals have the bottom dished to get this arch action. Sometimes struts brace the top of the lining. Concrete shapes often depend on the method used. Shot concrete is placed on horizontal, vertical, sloping, or curved surfaces. Poured concrete, with or without forms, is used in trapezoidal shape while, with forms, any shape desired can be obtained. Small covered channels with free-water surfaces are usually of concrete pipe. Cut-and-cover sections are usually nearly vertical in side wall, but sometimes circular or trapezoidal shapes are covered. Precast concrete "planks" are used to line sloping banks, especially on the outside of curves, to prevent erosion.

Wood.—Wood (other than in wooden flumes) is used to line trapezoidal sides and bottom of canals (pl. 10, *B*).

Brick.—Brick is not used to any extent in the United States for irrigation or power canals, but is still used to some extent in India. One system in lower Rio Grande Valley is using both brick and tile to line moderate-sized canals in rounded cross sections. Rough-finish coat covers the joints.

Metal.—Sometimes a channel is lined with sheet metal. (Spillway at El Vado Dam, N. Mex.) Usually a metal channel is a flume of some sort.

Rubble masonry.—Over most of the irrigated West rubble masonry has been largely replaced with concrete. However, in the lava flow terrain of the Northwest there are a few canals of laid-up lava rock, unlined or lined with a coat of cement mortar (pl. 24, *B*). Along the border between the United States and Mexico, the use of "mompos-teria" is quite feasible as the Mexican is an excellent worker in concrete and rubble masonry (pl. 20, *C*). This material is usually given about 1 to 1 or steeper slope in the side walls.

Most efficient section.—Important in the consideration of efficiency is the determination of the shape most economic from the hydraulic standpoint. For any shape, the best section is the one that gives a minimum value to $\frac{P}{A}$ (or a maximum value to R). If hard, unerodible materials are used, the most efficient channel is the semicircle. For the rectangle, the ratio of $b=2d$ is best.

In lining canals with concrete, of course a minimum quantity of concrete is desirable. This minimum is reached in the semicircle. However, for the larger canals construction features in most cases dictate the use of the trapezoid.

The maximum use of any available energy content $(d+h)$ is reached at critical flow. Taking shape into consideration the ultimate maximum efficiency is reached when critical flow is found in a semicircular channel. (An example is given (*57, p. 80*).)

While the most efficient cross section and shape seldom can be chosen, it is well to know what that shape and section are so that they may be approached, if not attained.

SINUOUS OR STREAMING FLOW

Nearly all canals are designed and operated at sinuous or turbulent flow.[18] This covers the streaming range from Reynold's critical flow to Bélanger's critical flow. Below Reynolds' critical point, the surface is glassy and movement almost imperceptible. Authorities agree that the loss of head is proportional to the first power of the mean velocity. This condition is, of course, not conducive to the primary purpose of conveyance of water. Oddly enough, at Bélanger's critical flow are found a glassy-like surface, a relatively transparent prism, and a nearly uniform velocity. If it could be assured and controlled, Bélanger's critical flow would be ideal for conveyance in channels that would not be scoured by the relatively high velocities. However, it is too sensitive to slight changes in canal surface to be used commercially, except for short distances.

If the channel is slightly smoother than assumed in design, the tendency is for the prism to race at velocities in the shooting stage, until checked by curvature or change in roughness. Thereupon the flow will change—usually through the hydraulic jump—to the streaming stage which is then slower than the normal stage, and the movement of the prism will quickly accelerate until normal stage is reached, whether faster or slower than critical flow.

[18] The writer suggests the more general use of "sinuous" flow, with "turbulent" being reserved for the violently agitated water quite commonly encountered in hydraulic engineering practice. As used by Reynolds, "turbulent" flow included everything above "glassy" flow which would include conditions found in most irrigation canals—a rather placid flow of 2 to 3 feet per second. If "sinuous" is used for such flow, "turbulent" can be used for a description more in keeping with such dictionary definitions as Webster's (1934 ed.): "Turbulent: Aroused to violent commotion; violently agitated; tumultous." Obviously the flow in most canals and gentle rivers does not meet this definition, yet they are turbulent according to Reynolds and this term has been carried in the literature of hydraulics to this day.

SHOOTING FLOW

(See Critical Depth, p. 4)

In canal chutes and reservoir spillways, velocities lie wholly in the zone of shooting, rapid or torrential flow, as it is variously called (pl. 8, *B*). Such flow is subject to phenomena now being critically studied as research problems. Some chutes and spillways develop "slug" flow in greater or lesser degree, and some do not, the reason for either condition not being clear (pl. 22, *A*). Again, some entrain air in appreciable quantities, thus swelling the volume so that the continuity equation $Q=AV$, no longer holds (pl. 22, *B*). So far as is known, the water prism with the swelled volume can be determined by the use of the same friction coefficient used for ordinary flow. However, the actual faster velocity is that computed with much lower values of the friction coefficient (*57, pp. 90–91*). On slopes that are obviously sharp inclines (pl. 22, *B*) the depth of water should not be taken in the vertical but in a plane normal to the bed of the channel.

The antithesis of the sharply inclined spillway chute sometimes holds just above the brink leading to the chute. Here the bed of the collecting channel leading to the spillway crest may be level or even have an adverse slope.

FLOW AROUND CURVES AND BENDS IN CANALS

Streaming flow around gentle curves in both earthen and lined canals results in a small amount of superelevation of surface on the outside (concave side) and depression on the inside, of the curve. Any scouring of an earthen channel takes place on the outside of the curve, and the raveled material is deposited on the inside (convex side) (pl. 10, *A*). For sharp curves or bends in a lined section (that cannot be easily scoured) the greatest velocities are near the inside (vortex flow?); the smoothest surface is on the outside, and the choppy water is on the inside of the bend (pl. 19, *B*). The water on the outside appears to roll up from below the surface and flow rapidly along the surface from the outside to midchannel, where it meets the edge of the choppy waves. The net result, in terms of design, is that about the same freeboard is required on both sides of the channel around bends or curves.

For sharp bends, the path of the flow on the inside of the curve is materially shorter than that on the outside. This, of course, gives a steeper gradient on the inside and makes more fall per unit length available for high velocities and for correspondingly heavy frictional losses.

For velocities faster than the critical (that is, for shooting flow) the direct forward velocity is faster than a velocity convertible to vortex flow, and it is not feasible to hold the prism within a trapezoidal channel on a curve of much sharpness. A vertical outside wall with a properly designed top may turn back the wave into the channel.

One obvious control can be effected by warping the whole channel so as to depress the floor on the inside and elevate it on the outside. The effects of curvature in canals may be itemized about as follows:

IN EARTH CHANNELS

New banks on the outside of moderate curves are easily eroded. They should be protected by brush or rock riprap. Curves sharp enough to class as "bends" are very difficult to hold in earth channels, except with riprap.

Mud and debris will be deposited on the inside of curves as they become available to the flow.

Velocities are so moderate in such canals that no provision need be made for difference of water surface elevations on the outside and inside of curves. The highest velocity is sometimes on the outside and sometimes on the inside.

IN LINED CHANNELS

For streaming flow, which occurs in most canals of such gentle slope as not to class as chutes, there appears to be a critical curvature.

Curves sharper than this critical can be classed as bends. For the same channel, this critical curvature may change with depths of water.

Flow around bends appears to be of two forms:

(1) On the inside of the bend the surface is depressed but choppy, the velocities are higher than on the outside; the flow is distinctly forward and color, string, or other flags held on a rod close to the side will hug the side of the channel. If the lining is formed by short straight reaches, separation of the prism and an eddy occur just below each angle point. Traveling debris on the bottom works toward the inside, as evidenced by a clean polished streak in many lined channels.

(2) On the outside of the bend the surface is elevated but smooth, the velocities in rectangular, circular, or trapezoidal shapes are slower than on the inside; the water appears to be rising rapidly from some point below the surface and rolling toward the center of the channel. Surface flow is from the outside toward the center (not toward the outside as many technical books and articles state). Flags held on rods at the outside take angles of 30° to 45° with the side, instead of hugging it as might be expected. On the outside, the flow appears to lack the straight forward features characterizing the inside flow. It surges up from below the surface as sand boils in a heavily silted river. Sometimes chips float aimlessly about, slowly working their way downstream.

FOR EXTREMELY SINUOUS LINED CANALS

The value of n used in design should be 0.001 to 0.003 higher than for the same conditions in a reasonably straight canal.

The surface is elevated on the outside and depressed on the inside, and the hydraulic grade line developed by averaging these two surface points for each station follows a very broken unpredictable path.

If the velocity head for the mean velocity at each station be added to the hydraulic grade line as computed above, the result is the energy grade line, usually called the energy line.

The energy line, even for this very sinuous canal, is remarkably straight, being without the steep dips that would be expected at each distinct bend.

This means that the local losses for the individual bends continue on downstream and blend with the normal loss. Thus it is not necessary to allow a special drop in energy for each individual bend, but merely to use a higher value of n throughout.

For streaming flow, no special freeboard on the outside appears necessary; the elevated but smooth water on the outside approximately balances the depressed but choppy water on the inside. However, some excess freeboard may be necessary both inside and outside.

Apparently, water surface warp reverses at the same point where alinement curves reverse.

For two close bends in the same direction the elevation of the outside and the depression on the inside are greater for the second curve.

The flow in a tangent below two close reversing curves is smoother than below curves following the same direction. This suggests that some point near the lower end of this tangent is a better place for a gaging station than one on a longer tangent following bends in the same direction. This is probably true of sinuous rivers as well as canals. (For more complete discussion see Scobey (*57*).)

FOR SHOOTING FLOW IN LINED CHANNELS

Curves and bends are to be avoided as much as possible. Little is known of the action of rapid flow in curved chutes. It was noticed that in one trapezoidal channel the water flowed up the steep outer side and left the bottom of the channel on the inside "dry." On the other hand, shooting flow approaching a sharp curve with nearly vertical outside wall passed through the hydraulic jump in twisted form, rolling the water prism back into the channel again.

Shooting flow may be converted to streaming flow before conveying the water around a sharp curve, or it may be modified by planting rough "plums" of boulders on the floor of the curve, with freeboard on both banks increased to care for the flow if the modification throws the prism into streaming flow.

For additional comments on the effects of curves and bends the reader is referred to the following authorities: Godfrey (*24*), Yarnell (*70, 71*), Mitchell and Harrold (*43*), Shepherd (*58*), Freeman (*21*), and Pearl (*49*).

EFFECTS OF MUDDY WATERS, AND AQUATIC GROWTHS

The statement is almost axiomatic that muddy waters do not have extensive aquatic growth out in the water prism and that clear waters in earthen channels are seldom free of such growths throughout the irrigation season.[19] However, intermittent flow of muddy water may not eradicate aquatic growth, even though muddy for 2 or 3 weeks at a time. In concrete-lined canals, silty waters sometimes form a coating on the concrete. If allowed to become thick, this coat forms a bed for the growth of various types of "moss." (Quotation marks are used as irrigation parlance includes many forms of growth that are not moss, strictly speaking.) Sometimes this combined mud-and-moss coat becomes thick and leathery. It then has a bunchy formation that loses the high carrying-capacity effect of a thin slick coat. Blow sand slides down the incline of concrete canal sides until it reaches the line where capillary water is effective, there becoming moist and accumulating in firm patches that do not scour out even in reasonably high velocities. This is particularly true if the waters of the canal are silty, the effect then being to add sticky colloidal muds to the sharp sand deposits.

Throughout the West the Russian-thistle, or tumbleweed, has a great influence on the reduction of capacity of canals. During a hard

[19] The difference is not attributable to climatic or other differences in the localities where the two waters have been observed; indeed, they may appear side by side. For instance, in the Rio Grande Valley, near El Paso, Tex., the clear water of drainage ditches, even though high in alkaline content, becomes choked with vegetation. On the other hand, the irrigation canals, carrying muddy but sweeter water, have relatively little vegetation.

wind, dry thistles from the last year's crop break off and roll along the roads and fields until stopped perhaps by an irrigation canal, where they not only reduce velocities by their own structure but also form the nucleus of mud islands. Extensive erosion takes place in the canal bed around each island.

Muddy waters like those of the Colorado River and Rio Grande reduce capacity by deposits that encroach from the two sides, forming berms up to a surface level close to that of the water (pl. 13, *B*). In this shallow water and rich mud, tules, arrowweed, and dense grasses take root and form deposits that cannot readily be scoured out but must be removed by mechanical means if the capacity of the canal is to be restored. Usually the deposit of mud on the bottom between the berms is relatively thin.

The growth of vegetation in clear-water canals depends much on water temperatures. In localities where the supply is clear mountain water, fresh from melting snow, the effect of vegetation is at a minimum. However, those same waters, after miles of transport, become like the waters of the Southwest, warm and limpid. In earth channels and at ordinary velocities these waters are particularly subject to the vegetation hazard. Relatively high velocities, say 3 to 4 feet per second, discourage plant growth; so does deep water; i. e., 3 feet or more.

Canals carrying muddy waters should be designed for high velocities even though drop structures may later have to be installed. This is particularly true of new systems not expected to operate at full capacity for many years. If the canals should be operated at partial capacity, with low velocities, the mud deposits would form typical meandering flow on a bed built up within the larger canal bed (pl. 23, *B*). When the full-design capacity is required, say several years in the future, the banks will be so well seasoned that they can withstand a much higher velocity than was believed possible only a few years earlier.

A shaded canal is seldom afflicted with aquatic growths. However, trees use water consumptively.

Not all acquired conditions are adverse in effect. It has long been known that thin coatings of slick mud improve the capacity of rough concrete or graveled canal beds, unifying the surface over local holes and humps. Canals originally excavated in glacial drift and morainal material are relatively low in capacity and high in seepage loss owing to the predominance of cobble boulders of all sizes. Artificially muddied water has often been used in such canals to lessen the seepage and grade the material of the canal bed, making it smoother and increasing its capacity.

For additional discussion of canal capacity as influenced by aquatic growth and muddy waters, the reader is referred to the following authorities: Lippincott (*40*), Kennedy (*32*), Finley (*16*), Pacific Electric Association (*8*), Burkholder (*7*), report of the City of Los Angeles (*41*), Grunsky (*25*), Lindley (*37*), Buckley (*6*), Haltom (*26*), Dibble and Parry (*13*), and Etcheverry (*15*).

PERMISSIBLE VELOCITIES

In canals having conveyance as their sole function, necessity to conserve slope usually prescribes velocities in either earth or concrete channels that are in streaming stage of flow. In concrete and other

tough material, permissible velocities are only of moment when the channel is designed as a chute—essentially a drop structure—to lower a body of water from one general elevation to another. The mean velocity of a canal section is a direct factor in the amount of water it carries, since $Q=AV$. From a cost standpoint alone, to get the maximum capacity for the minimum cross section, of course the highest feasible velocity is desired. In earthen channels this is usually under 3½ feet per second; in gravel, from 4 to 7 feet; and in concrete, metal, and wood, usually under 6 to 8 feet per second (pl. 20, *B*). Other factors being equal, a canal should be designed for a velocity approaching that permissible for the material of the canal and the type of water to be conveyed. The other factors that may operate against such velocities are discussed below.

1. It may be necessary to conserve slope in order to command a larger area. This would reduce the velocity for a given material.
2. It may be necessary to prevent silting and scouring in a given channel conveying muddy waters. This may dictate the use of recent data for nonsilting-nonscouring velocities (*32*, *35*). These are somewhat below those usually thought of as permissible.
3. It may be necessary to use a rather low velocity to induce precipitation of silt from the water in order to reduce seepage losses, especially in a new canal in a well-settled country, where seepage water might do expensive damage.

In many of the older books on irrigation and hydraulics permissible velocities were confused with transporting velocities. It is now known that the two are almost paradoxical—that the material most easily transported may be relatively hard to scour. That is, old seasoned canal beds with slick silty banks are not easily eroded, but these silty particles, when raveled off the bank, easily remain in suspension in the water and are most easily carried. In fact, if transported as colloidal matter in a dispersed state, they may even stay in suspension without appreciable velocity. On the other hand, clean sugar sand is so easily set in motion that a clean sandy bed is continually changing local formation; yet this material may require a much higher velocity to pick it up and transport it downstream.

Table 2 was developed by the author from material submitted to the special committee on irrigation hydraulics of the American Society of Civil Engineers (*20*). The contributors of these data were operators of irrigation systems throughout the West and were well informed as to the action of waters in their particular canals. As developed originally, this table was circulated for discussion throughout the membership of the society and received but little, if any, adverse comment. It has since then been freely copied into engineering literature.[20]

[20] The term "effective colloids" as used in this table, should be explained. Colloidal muds, to have a maximum effect in cementing and binding the bed of a canal, should have been deposited from the dispersed rather than the flocculated state. In the dispersed state they are very fine, are transported by any velocity whatever and in many cases remain in suspension in still water for a long time. Likewise, they are negatively charged electrically, but will ground out when brought into contact with a containing conduit. Deposits in a pipe line, resulting from this process, are evenly distributed over the periphery of the pipe—as much on the top as on the bottom. Obviously, this is entirely different from sedimentation, where the water clears wholly by gravity settlement. A description of dispersed and flocculated silts from the Colorado River has been given by Breazeale (*5*). The velocities given in table 2 are based on well-seasoned canals. In irrigation practice maximum velocities usually develop slowly, with the growth of the settlement on the project. For less than maximum capacity the mean velocity is materially less and scouring is not likely to be serious. However, the irrigation demand in the early days of operation may be so much less than canal capacity that the resulting velocities do not prevent excessive silt deposits and may develop a meandering stream on the new bed formed by them (pl. 23, *B*). Even a small irrigation demand may be too much for a new canal on sharp bends with consequent excessive erosion on the outside and silt deposits on the inside of the curves (pl. 23, *A*)

TABLE 2.—*Permissible canal velocities*

Original material excavated for canal	Velocity after aging, of canals carrying— Clear water, no detritus	Water transporting colloidal silts	Water transporting noncolloidal silts, sands, gravels, or rock fragments
1	2	3	4
	Feet per second	*Feet per second*	*Feet per second*
Fine sand (noncolloidal)	1.50	2.50	1.50
Sandy loam (when noncolloidal)	1.75	2.50	2.00
Silt loam (when noncolloidal)	2.00	3.00	2.00
Alluvial silts when noncolloidal	2.00	3.50	2.00
Ordinary firm loam	2.50	3.50	2.25
Volcanic ash soil	2.50	3.50	2.00
Fine gravel	2.50	5.00	3.75
Stiff clay (very colloidal)	3.75	5.00	3.00
Graded, loam to cobbles, when noncolloidal	3.75	5.00	5.00
Alluvial silts when colloidal	3.75	5.00	3.00
Graded, silt to cobbles, when colloidal	4.00	5.50	5.00
Coarse gravel (noncolloidal)	4.00	6.00	6.50
Cobbles and shingle	5.00	5.50	6.50
Shales and hardpans	6.00	6.00	5.00

Supplementing table 2 are certain definite findings, which should be fully understood before figures from the table are applied.

FOR EARTHEN CANALS

1. All velocities should be less than Bélanger's critical velocity; i. e., all flow should be in streaming stage.

2. Permissible velocities are usually much higher than transporting velocities for the same materials. That is, in a well-seasoned canal, a higher velocity is reached before the bed is broken and eroded than is necessary to transport the eroded particles after they become detached.

3. In a well-seasoned earth canal, the bed is usually composed of particles of various sizes, compacted, with voids in each size of material filled with smaller particles until the result is a tough, sometimes slick coat. This coat is not easily broken if kept relatively free of foreign objects, such as large rocks or anchored tumbleweeds which start eddies that break the bed and lead rapidly to serious erosion.

4. Dispersed colloids, in either the material of the canal bed or the water conveyed by it, or in both, tend to cement particles of silt, sand, and gravel in such a way as to make them resist erosive effects.

5. Flocculated colloids are useful in filling interstices in more open material, but they do not have the high cementing properties of those in a dispersed state.

6. The grading of material, running from very fine to coarse, coupled with the adhesion brought about by the sticky colloids, make possible high mean velocities without any appreciable scouring effect.

7. Canals when new usually flow over many types of material, owing to the variation in the soils encountered. A relatively homogeneous bed is highly desirable; therefore artificial silting may be used if natural muddy waters cannot be relied on during any appreciable part of the season.

8. Irrigation canals usually require check structures for the control of deliveries to laterals on either side. These checks can be used to retard velocities in easily eroded parts of the canal or to induce the settlement of muds artificially placed in the canal for this purpose, perhaps several miles above the checks.

9. The use of checks, whether necessary for irrigation deliveries or not, allows higher velocities than might otherwise be permissible. By this means, velocities can always be reduced, but it is very difficult or impossible to increase the velocities if too low a standard is set in the beginning.

10. If the canal be for hydroelectric power, municipal, or other use requiring that it be put under approximately full capacity as soon as feasible, a lower velocity should be chosen or curves and other critical points protected by riprap in some form. (Item 9 may also receive consideration.)

11. Sometimes earth channels can be built for initial use on canal systems, with the expectation of obtaining the eventual higher capacity by lining the canals with concrete. Fortunately, the gradients necessary for flow in the earth canals are also those conducive to a good operative velocity in a lined section. For usual irrigation canals and laterals the capacity of a lined section is about 1.5 times that of an unlined canal.

12. Early erosion, under, say, 4-foot velocities, may be checked by blanketing the canal bottom with bank-run gravel from which the sand has been removed. Silt remaining in the gravel, or silt in the water, eventually makes a smooth coat not eroded by velocities under about 5 feet per second, with a value of n around 0.022.

FOR COBBLE-BOTTOM DITCHES

Most unlined canals leading from the mouths of canyons in the Mountain States cross debris cones or glacial moraines. While such canals are originally excavated in "earth," it soon develops that the finer materials are carried away by the water and a distinct type of cobble armor becomes characteristic of the channel bed (pl. 22, C). The mountain streams carry muddy waters during the spring and after summer storms. Most of the irrigation season they are crystal clear. After this armor develops, a high velocity is permissible. However, this possibility is offset in new canals by the need of precipitating any silt that flows in them, as they nearly always have excessive seepage losses for many years after they are built. These canals take on many of the characteristics of the streams from which they are diverted. There is almost no downward cutting but lateral erosion is effected by washing out the fines and dropping the cobble plums onto the bed of the canal. After the canal has become well bedded with graded material, the high permissible velocity may be attained by careful removal of stray cobbles as they accumulate either by rolling in from above the upper bank of the canal or as the result of lateral erosion from one bank or the other.

FOR CONCRETE-LINED CANALS

Many of the older treatises on hydraulics list about 15 feet per second as the upper limit for the hardest rocks. With many examples representing extensive experience it is known that very high velocities can be used in modern well-made hard concrete. Canal chutes with velocities from 20 to 40 feet per second are common; scattered examples show even up to 80 feet per second. The spillway tunnels at Boulder Dam are designed for velocities of 150 feet per second. For any particular flood, such spillways are in operation for a few days at most.

In concrete all velocities that might class as the upper permissible are faster that Bélanger's critical velocity and are thus in the zone of shooting flow.

Before allowing water flowing at such velocities to come to the end of a concrete chute and be discharged into an earth canal or a stream, it is well to change the velocity through the hydraulic jump to a subcritical velocity. This will dissipate much of its energy and will not subject the banks below the chute to excessive erosion.

In straight channels with clear water, in gliding contact with the bed, there is no practical limit to velocities. Sharp abrasives may erode cement and fine sand (pl. 4, *D*). The resulting pebbly surface changes little.

In concrete chutes, dense carpets of "moss" are quite common, thus keeping the swift waters from direct contact with the channel proper. In floodways, dense Bermuda grass will withstand 6- to 8-foot velocities.

High-velocity flow is often associated with so-called slug flow, which causes a highly uneven turbulence in the outlet pool and excessive erosion (pl. 22, *A*). Just how it is caused and when such flow is to be expected is not known.

For additional discussion on permissible velocities the reader is referred to the following authorities: Fortier and Scobey (*20*) Davis (*11*), Paul (*48*), and Etcheverry (*15*).

REGIONAL CHARACTERISTICS INFLUENCING CANAL CAPACITY

Climate, terrain, geology, waters, winds, insect life, and aquatic growths vary so widely throughout the West that certain characteristics can be identified with localities.

In all the Western States, slimes (pl. 4, *C*) and aquatic growths can be expected in clear-water canals, while only in the extreme South and Southwest are heavy arrowweed, tamarisk, willows, tules, and cattails found growing along the edge of silty-water canals.

Canals served by streams from the lava beds of Washington, Oregon, and Idaho are usually clear, except for the short periods just after heavy showers. Various "moss" growths reduce capacity to a marked degree, particularly in July and August. Nearly any growth in the water prism is called moss, though most of the types commonly found are not true mosses (pl. 12, *A*). A large part of the maintenance funds of canals is devoted to the eradication of these growths. Likewise, in this region, certain insects pass part of their life cycles in the water. The caddisfly, for example, lives through the larval and pupal stages in a case composed of small pebbles, pine needles, and other trash, cemented into a twiglike structure about three-

sixteenths of an inch thick and from 1 to 2 inches long (pl. 4, *A*). These cases and algae growths that appear to exist under the same conditions can easily raise the value of n from 0.013 to 0.016 or more in a season, becoming a maximum at the season of greatest water demand. Various methods of combating such growths have been tried (*63*, *65*, *66*).

Canals in lava terrain usually carry sharp ravelings of basalt that remove the finer elements of concrete, leaving a rough bed with a value of n materially higher than that of the original concrete. On very steep chutes in such country the abrasive is usually whirled up into the water prism and does not appear effective in keeping down the "moss" growths characteristic of high-velocity chutes.

In Montana, Wyoming, and the Dakotas sweetclover and mustard grow densely along the edge of the water and drag in the water prism. In southern California and Arizona water grass and Johnson grass are great offenders in a similar way. In lower Rio Grande Valley of Texas, Bermuda grass works down from the canal banks and drags in the water; it also grows quickly in the bed of any lateral without water for a few days, since there is nearly always a shallow deposit of silt, even in the lined sections. Willows and other large trees in California and Arizona send dense root masses out into the water prism—either clear or silty—not only consuming large quantities of water but also forming a mat that fills with silt and is difficult to remove. Such trees prevent the use of machinery in cleaning canals and thus, while they are beautiful and shady, have little or no economic place along a canal bank. In New Mexico, Colorado, Wyoming, and Utah, willow brush and cottonwood usually establish themselves along a ditch bank if unmolested. In Montana long streamers of horsetail moss reduce capacities as much as 60 to 75 percent. This so-called moss is also common in Utah, Idaho, and Colorado. Aquatic vegetation is not limited to the States mentioned but the types are particularly active in the States named.

Winds have great influence on capacity. In parts of Wyoming, Montana, Nebraska, Nevada, eastern Colorado, Imperial Valley of California, and Yuma Valley of Arizona, strong winds blow sand into the canals, hold back the flow if blowing upstream, and sometimes fill a canal with tumbleweeds of Russian-thistle or other plants. Russian-thistles are particularly troublesome in Nevada during June. In lower Rio Grande Valley in Texas, the wind tends to blow fine sand into lined sections of canals. It is found above the water, in shallow drifts, that become moist and hard from water drawn up by capillarity from the canal. A canal seriously threatened by blowing sand may be protected by low fences of one or two 12-inch boards placed several feet from the banks. These boards cause sand drifts to pile up like snow on the leeward side, whence it may be removed or smoothed down after the storm. Where sand is anticipated, a reasonably high velocity may be used to keep it moving down the canal rather than filling the available prism. Of course, some of the semiaquatic growths along canal banks have the saving virtue of stopping some of the wind troubles.

In Colorado, all canals from mountain streams are clear except just after rains. In the warmer parts moss and watercress are common. Leaving Colorado, the main stem of Rio Grande becomes turbid as it flows out into the middle Rio Grande Valley near Espanola, N. Mex.

This condition becomes worse until the stream enters Elephant Butte Reservoir. From this settling storage it issues as clear water, except during certain floods from the Puerco. However, it picks up more or less silt and soon becomes muddy again, but is not so heavily laden as just above the reservoir.

In Texas irrigation water from Rio Grande is very muddy as a rule. Water used in Salt River Valley of Arizona is not clear nor is it so heavy as either Rio Grande or Colorado River water. Diversion structures now being constructed (1937) on lower Colorado River provide for desilting the water.

In Wyoming and Colorado, the Green and Colorado Rivers are very clear in their upper reaches and only moderately muddy up to their junction in Utah. Below that point, the Colorado becomes very heavily laden with silt, in both dispersed and colloidal states. Most of this mud settles out in Lake Mead. Generally clear water will leave this reservoir but will pick up more sand and a little silt until again cleared in the reservoir above Parker Dam. Heavy diversions for irrigation will be made below this dam. In recognition of the fact that the river will again be loaded with bed and suspended silts, an elaborate structure is being built at the Imperial heading to rid the water of much of this burden. Thus the irrigation canals of Yuma and Imperial Valleys will have much less mud and sand to contend with than formerly. However, it may be necessary to make changes in canal design for these areas. Formerly canal velocities of 5 to 6 feet per second were not considered excessive, but with relatively clear water the limiting velocities may have to be reduced, with corresponding reduction in capacity for the same size canal. For old canals, this reduction will probably be effected by the construction of drop structures; new canals will be designed for lower velocities. Likewise the clearer water will increase aquatic growths.

Canals irrigating or crossing the gravel cones immediately below the mouths of mountain canyons in Montana, Wyoming, Colorado, and Utah, and some of the glacier streams of Washington and Oregon, are usually characterized by steep, grassy banks and cobble bottoms (pl. 24, *A*). When well aged, such beds become graded from fine to coarse gravels, with each rock well bedded. Such terrain is often on benchland above the general river bottom. When new, seepage losses are excessive but some canals from 60 to 80 years old are said to be remarkably tight, provided the parent stream carries fine silt a portion of the time or artificial silting is effected to fill the voids. Sometimes such canals start with values of n of about 0.022 and then lose capacity as more and more cobblestones wash from the matrix of the banks or roll in from above until the value approaches 0.030; but the accumulation of graded material and silt finally makes a fairly smooth bed and the value of n returns to around 0.022.

The soils of western Nebraska are particularly adapted to hard, clean canal sections with very low values of n (even below 0.016). Selection of too high a value for design may lead to excessive velocities and consequent erosion.

In Arizona, diversions from the Gila, although below Coolidge Dam, sometimes run high in silty muds that rapidly encroach on tanal capacity unless the canal can be operated at sustaining velocities (pl. 23, *B*).

For additional discussion of canal capacity as affected by local conditions, the reader is referred to the following authorities: Hopson (*27*), Lippincott (*40*), Taylor (*63*), Reclamation Service (*65*, *66*). The older volumes of Reclamation Record mention many such examples in maintenance and operation notes.

HYDRAULIC ROUGHNESS

HYDRAULIC ROUGHNESS INCLUDES MORE THAN ROUGHNESS OF CHANNEL SURFACE

The rate of flow in a new, straight, uniform channel, conveying clear water, is largely determined by the roughness of the channel surface, e. g., of earth, concrete, wood, metal, or other material. In ordinary field use channels do not remain new; they are seldom straight; flow against or with prevailing winds; do or do not acquire growths of insect life, moss, tules, cattails and the like; and carry water that contains more or less silt, sand, and gravel, or are subject to inflow of such materials from their banks or blown in by winds.

All these secondary influences set up conditions that may have much greater influence on the capacity than the roughness of the original channel. In the design of canals, it is this massed influence of all conditions, both on the channel surface itself and out in the water prism, that must be anticipated in the selection of a value of n or other roughness coefficient (*35*, *p. 304*). The values of n as found in field research are due to this hydraulic roughness rather than to any surface roughness only. Seldom can the separate losses due to each of many conditions other than channel surface be evaluated.

Since there is always friction, even in the best of channels, it is best to think of the conditions in terms of degrees of roughness rather than the degrees of smoothness. Thus, the rougher (hydraulically) the channel, the larger the value of n.

For any material of construction, the hydraulic roughness is a minimum, the velocities and hence the capacity a maximum, if precautions are taken to—

1. Specify and obtain a good grade of original surface, if of concrete it should be hard as well as smooth.
2. Get as favorable alinement as is feasible at reasonable expense. Gentle curvature is usually unavoidable and its influence is sometimes difficult to locate in terms of values of n. In other words, losses due to such curvature are not excessive. Sharp curvature and bends have definite extra losses and should be provided for.
3. Reduce the mud content in the canal water to a minimum, especially the bed load. It may be advisable to retain silty muds to reduce percolation losses and the capacity of cobble bottom channels will be improved by such muds in filling spaces between rocks.
4. As water is cleared, aquatic growths increase. Maintenance operations should provide for the continuous reduction of such growths.

For discussion of roughness in addition to that of the containing channel surface, the reader is referred to the following authorities: Buckley (*6*), Dibble and Parry (*13*), Ellis (*14*), Finley (*16*), Grunsky (*25*), Haltom (*26*), Hopson (*27*), Lippincott (*40*), Taylor (*63*), (*65*), and Etcheverry (*15*).

CHANNEL SURFACE ROUGHNESS AFFECTS THE FLOW

When the cross section of the usual flowing canal is explored with a current meter, it is noticed that the velocities increase from the sides toward the center and from the surface and bottom toward a point approximately three-tenths of the water depth below the surface. These increases take place in fairly smooth curves without shearing offsets, except right at the channel surface.[21]

If the velocities at the side and bottom are excessively retarded by a very rough channel, then the whole curve of velocities is held back across the water prism (fig. 3). Likewise, the curve from the water surface down may be greatly retarded by wind movement upstream. The author has seen flow almost stopped in shallow canals during heavy adverse winds. Of course, a downstream wind will increase the velocities by pulling forward the surface ends of the vertical velocity curves. Thus canals flowing with or against prevailing wind directions should be designed with this factor in mind.

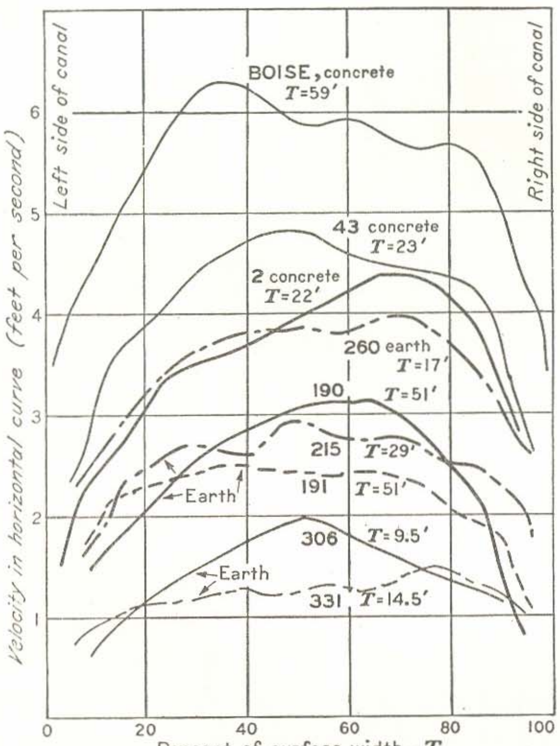


FIGURE 3.—Typical horizontal velocity curves, showing trends toward bank velocities for canals of various values of n. If smooth surfaces increase the bank velocities then the whole velocity curve is increased. It is not possible to hold a current meter at the extreme sides of lined channels, hence the trend only can be shown. The number of the curve corresponds to the reference number in table 1.

PRIMARY AND SECONDARY ROUGHNESS

To get the best capacity results, local surfaces should be uniform as regards the material, and large surfaces should be free of undulations and sudden changes in configuration (pl. 4, *B*). A material that lends itself to one kind of roughness in local areas and another kind in larger areas develops primary and secondary roughness. To illustrate: Corrugated metal is made of a smooth material—the primary surface—but is quite rough hydraulically owing to the fluting of the corrugations, a secondary roughness. Shot concrete, when cast without forms and untouched, is rough to the hand and undulating of surface, thus having both primary and secondary roughness in high degree (pls. 3, *C* and 21, *A*). Some plaster coats are similar in texture (pl. 4, *B*). Concrete cast against forms, on the other hand, may have many small air pockets and be quite rough in primary surface. However, if smooth rigid forms are used, the resulting surface will be without undulations, and thus the secondary surface may be excellent and overshadow the local roughnesses. Secondary roughness affects the capacity of a channel by inducing slashing eddies that completely annul the good effects of a smooth primary surface. A sandy canal bed may develop even three types of roughness. When bedded down, such a bottom is quite smooth; the equal of a good earth channel. Such sand usually requires a small admixture of clayey muds. At the saltation stage a rough bottom of small traveling dunes reduces capacity both by loss of channel space but also by higher values of n. On the other hand, large canals and rivers in sandy beds sometimes acquire a bottom with many deep temporary pockets where sand boils have "exploded" (pl. 23, *C*). The pockets so formed soon fill with other bed material and other pockets are developed.

The All-American Canal in southern California would probably have been of this type if a sandy channel, as excavated, had been used. However, this type of channel was made oversize and lined with a refill of selected earth so compacted that entirely different conditions will prevail, and the pockets so characteristic of sand beds will not develop.

Secondary roughness has often been overlooked in choosing a value of n for design purposes.

[21] In a large channel it is difficult to determine what the horizontal velocity curves are doing just as they reach the side of the water prism. In careful meter measurements on Tiger Creek flume, in California (a channel 14 feet wide and approaching the rectangle in shape) the mean horizontal velocity curve was projected 0.4 foot until it reached the concrete sides of the flume. For a mean velocity of 7.19 feet per second the maximum was 7.80 and slightly to one side of the center; the minimum at the left side was 5.52, and on the right side 5.88 feet per second. This measurement was in a reach of canal for which the value of Kutter's n was determined to be 0.0118, indicating something akin to shearing right at the smooth concrete wall. Thereafter the curve of velocities gradually reached a maximum and then diminished as the opposite side was approached. Had the channel sides been slightly roughened, the extreme side velocities would have been less and then the whole horizontal velocity curve would have been dragged back so that the same quantity of flow Q, would have required a larger cross section.

OTHER CONDITIONS THAT AFFECT THE FLOW

Were hydraulic roughness confined to the surface of the containing channel, the range of surfaces would begin at the equivalent of plate glass or celluloid and end with the roughest of rock cuts and boulder-strewn torrents. A surface approaching that of plate glass is attained in some of the best workmanship in concrete. Such a surface might be represented in an open channel by Kutter's $n=0.010$. Very small channels in glass, smooth cement, or celluloid attain even 0.008 for n. At the other extreme would lie the rough rock cut with a value of 0.040 for actual cross section and, say, 0.035 for the "paper" section on which the cut is based. However, values of n even beyond 0.100 are reached. In any given canal, values above or below those attributable to the containing surface are induced in many distinct ways. Moss, tules, cattails and other water-loving plants especially reduce capacity. Scattered patches of rock debris, shifting sand, and the like not only reduce the capacity by dragging back the velocity filaments of current but also by diminishing the area of the water prism. Likewise, excessive curvature, channel constrictions at bridges, checks, and other operation structures all cut down the general capacity of the channel, unless complete allowances for their separate losses have been included in the various steps of fall allotted to each portion of the canal.

It is known that a smooth coating of fine silty mud will slick up a rough concrete surface and improve its capacity. Of course, beyond a certain amount of this action the capacity would drop, owing to the throttling effect on the channel area.

The preceding paragraphs explain why hydraulic friction controlling the capacity of any channel covers much more than the skin friction of contact between the channel surface and the water prism, and also between the water surface and the air. Too often the designer contemplates the channel surface only, without regard to the many other influences that may have greater effect on the capacity than the condition of the channel surface.

ESTIMATE DIAGRAM

Figure 4 gives a solution for general problems involving the Kutter formula. The use is best explained by an example. The dashed lines show that in a channel with hydraulic radius, $R=2.6$ and with an assumed value of $n=0.015$ and a slope of 0.00125 the velocity will be about 6.7 feet per second. The quantity of flow, Q, is then equal to AV. This diagram can be used for design of canal sections of any shape. For very flat slopes interpolate between guide lines, which are split for divergent values of n.

RECOMMENDATIONS FOR VALUES OF *n* FOR DIFFERENT KINDS OF CANALS

While the following discussion is in terms of the various materials of which the canals are constructed and thereafter known, it is fully appreciated that the greatest variations from standard values are due to influences that may not identify themselves with any particular materials of construction. Moreover, these influences do not always appear in the form of surface roughness. In these facts lies the greatest difficulty hampering the development of retardation factors for a few categories of roughness and then placing all assumed variations within one of these categories according to their appearance compared with the known criteria. For example, a clean, rough lining of shot concrete untouched by trowel or other smoothing device, may have exactly the same roughness factor as a smooth concrete channel with a well-developed growth of moss in the water prism. In the one case an excellent footing is obtained, the surface is very rough to the hand and an obvious lack of refinement exists. In the other, little footing is obtainable, the surface may be slick and smooth to the hand and there is little or no appearance of surface roughness. Yet the capacities are both obviously below par; in other words, roughness may not be dimensional.

The cause of the hydraulic roughness may be quite different in the flowing prism of water from the condition appearing when the water has been turned out of the canal. Such circumstances make it impossible to evaluate roughness coefficients except in an empirical way. If certain surfaces could be established as standard (say comparable with the seven primary colors) they could perhaps be definitely evaluated in a laboratory. There would still be the many variations of surface and other conditions causing hydraulic roughness comparable with the innumerable shades of colors, and these would be more common than the standards originally tested.

VALUES OF *n* FOR EARTH CANALS

	Basic value, *n*
Moderate size canals and laterals	0. 0225
Very large canals	. 020
Very small canals and ditches	. 025

The basic values given above are extensively used for design purposes in the United States. However, there may be wide variations

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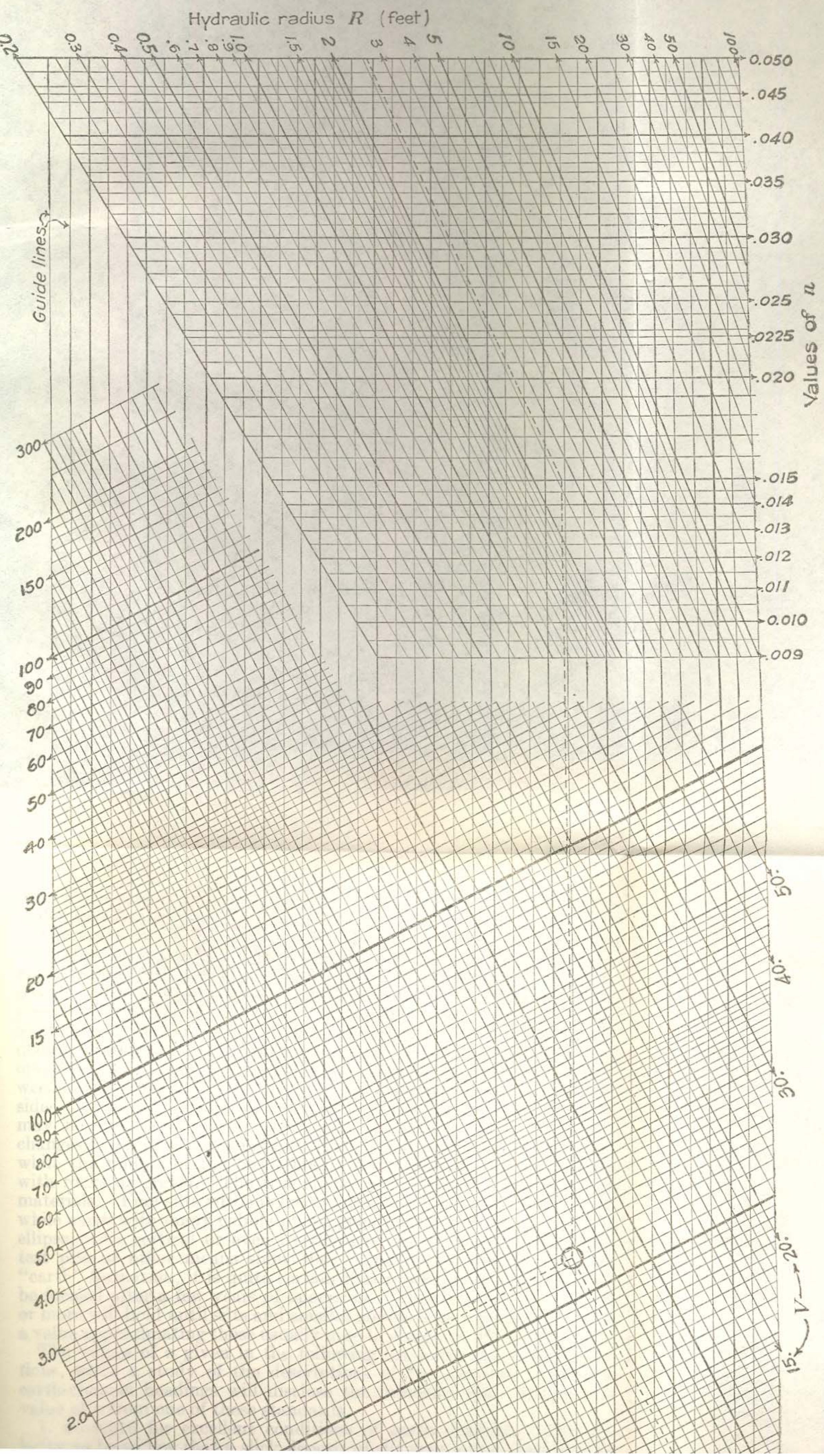
Hydraulic radius R (feet)
0.2
0.3
0.4
0.5
.6
.7
.8
.9
1.0
1.5
2
3
4
5
10
15
20
30
40
50
100
0.050
.045
.040
.035
.030
.025
.0225
.020
.015
.014
.013
.012
.011
0.010
.009
Values of n
Guide lines
300
200
150
100
90
80
70
60
50
40
30
20
15
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
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40
30
20
15
V

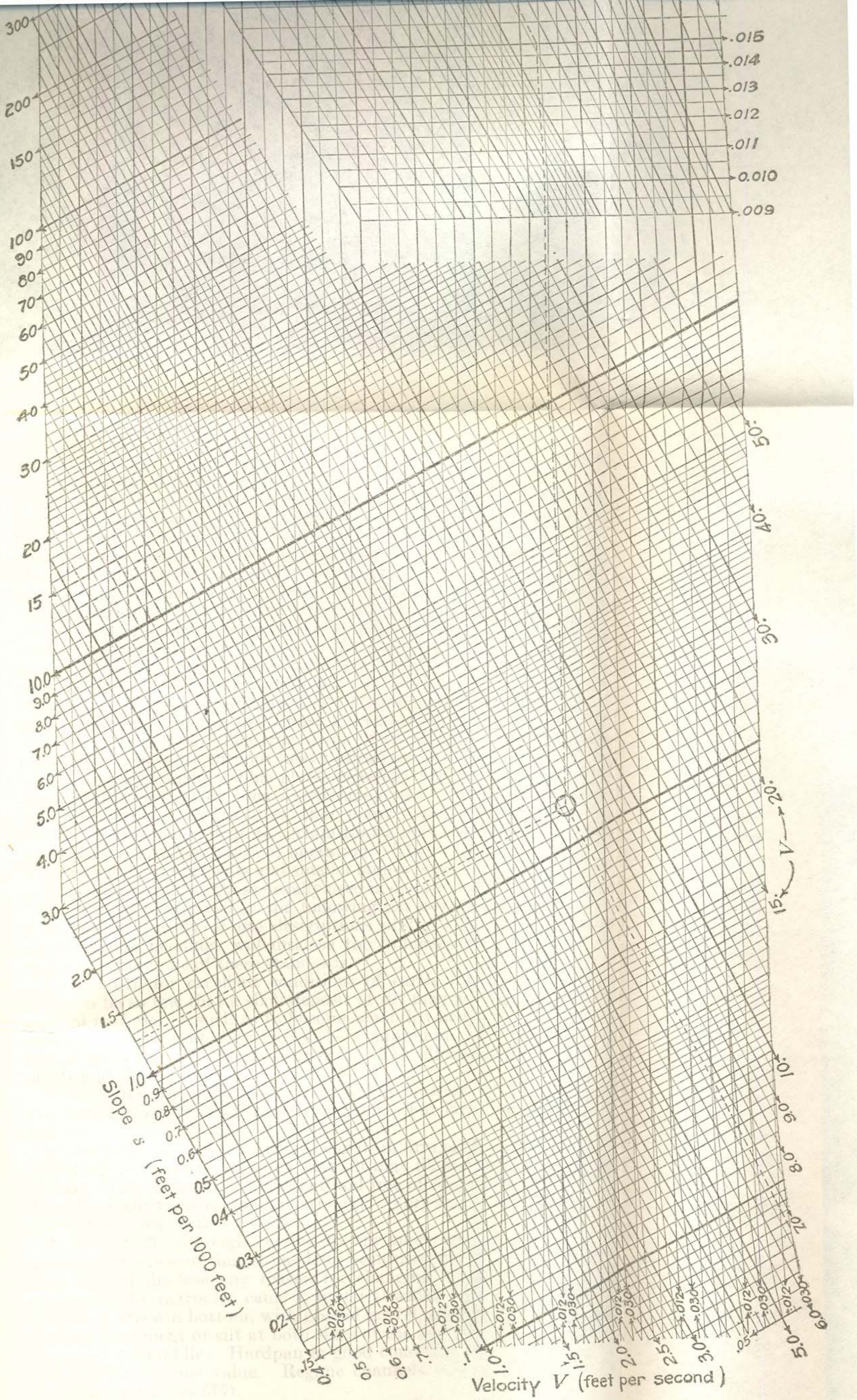

FIGURE 4.—Diagram for the solution of general problems involving the Kutter formula. From the intersection of R and n follow the guide lines to the intersection of S and V, or from the intersection of S and V follow the guide lines to the intersection R and n. For example shown in dashed lines, see page 66.

from these values, and the conditions that lead to them are indicated under specified groups.

The value of n for earth channels extends over a far greater range than that typifying any other material. More complex conditions, more permutations and combinations of conditions exist than are possible in a channel not erodible at reasonable velocities. If kept clean, concrete, wood, or steel must maintain about the same cross section, uniform as a rule. Earth, on the other hand, may form a definite periphery of a channel when new, but after a few years of operation the character of the boundary has entirely changed. Grass, weeds, and fibrous roots may form the material for nearly vertical sides (*35, p. 233*), while the bottom may silt up or scour deeper; it may be smooth or deeply pocketed. A distinct trapezoidal form changes to a segment of an ellipse, silt depositing in the lower corners, while the middle of the bed remains unchanged or becomes strewn with rocks or gravel. The true shape of the ellipse depends on the materials. Fine silts without heavy abrasives approach a half circle, while earth with many cobble plums generally shapes into a very flat ellipse, with vertical or even overhanging sides (*35, p. 233*): When test pits or other means show that many cobblestones are in the "earth" formation, it is quite certain that a cobble-bottom canal will be formed after a few years of operation. Then a value of $n=0.026$ or more will hold until silts and smaller stones form a graded bed with a value approximating those usually used in design ($n=0.020$ to 0.025).

The values of n given in the following list cover the usual conditions, while a study of the descriptions of the channels under the earth-channel headings will disclose the influence that changes the value of n from one of these standards.

1. $n=0.016$ for excellent conditions of earth channels. The velocity to be so low that a slick deposit of silt may accumulate, or the natural material be such as to become smooth when wet. The influence of vegetation at the edges to be a minimum. The water to be free of moss and other aquatic growth. The alinement to be free of bends and sharp curves.

2. $n=0.018$ for conditions intermediate between types 1 and 3. For volcanic-ash soils with no vegetation. For large canals in very fine silt.

3. $n=0.020$ for well-constructed canals in firm earth or fine, packed gravel where velocities are such that silt may fill the interstices in the gravel. The banks to be clean-cut and free of disturbing vegetation. The alinement to be reasonably straight. Very large canals of type 4 may be designed with $n=0.020$.

4. $n=0.0225$, although carried to one more significant figure, is given for the reason that it has long been used for this type and the tests do not disclose any reason for changing. This value for the average well-constructed canal in material which will eventually have a medium-smooth bottom, with graded gravel, grass on the edges and average alignment or silt at both sides of the bed and a few scattered stones in the middle. Hardpan in good condition, clay and lava-ash soil take about this value. Regime channels take about $n=0.0225$ for design purposes (*35*).

5. $n=0.025$ for canals where the retarding influence of moss, growths of dense grass near the edges, or scattered cobbles begins to show. The value of n in earth channels where the maintenance is

neglected commences at this value and rapidly goes up. This is good value to use in the design of small head ditches or a small ditch to serve but one or two farms. Also for canals in a hilly terrain, where the upper bank may be wholly in cut, thus making the canal liable to much debris rolling down the hillside above. Such canals should be cleaned at least once a year or they will not keep in condition that yields a value as favorable as 0.025. This value was much used from 1900 to 1910 for canal design. Since then 0.0225 has been adopted for the same conditions.

6. $n=0.030$ for canals subject to heavy growths of moss or other aquatic plants. Banks irregular or overhanging, with dense rootlets. Bottom covered with large fragments of rock, or bed badly pitted by erosion. Values of n between 0.025 and 0.030 also cover the condition where the velocity is so high that cobbles are kept clean and unpacked in the center of the canal, but silt deposits near the sides. A complete covering of coarse gravel without firm bedding takes about this value. Such canals usually constructed in earth conglomerate. For large floodway channels, well maintained.

7. $n=0.035$ for canals about 50 percent choked with moss growth. For flood channels that will not receive continuous maintenance, say with a dense Bermuda-grass carpet.

8. $n=0.040$ for canals badly choked with moss or heavy growths. For canals in which scattered large cobbles and boulders will collect; approaching a mountain stream in clean loose cobbles.

Floodways poorly maintained go from 0.050 up.

VALUES OF n FOR CONCRETE LININGS

Where water is highly valuable, the attainment of a given capacity essential, or the loss of water detrimental, concrete lining is generally used. It may be a coat of cement-and-sand plaster, say three-quarters of an inch thick, applied to a moist, neatly trimmed earthen bank (pl. 7, *C*). It is usually of reinforced concrete developed by dumping, pouring, or pumping more or less moist concrete mixtures into panels and bringing the surface to a predetermined location by means of screeds, trowels, or movable smoothing platforms (pl. 1, *A*). The panels are usually separated by expansion joints. Concrete may also be shot by compressed air as a mixture of cement, sand, and water against a moist, trimmed earth bank (pl. 2, *A*); against wood or steel forms; as a body matrix, filling rubble walls to a relatively even surface, or as a coating to fix the surface of a rock cut and prevent weathering and leakage (pl. 21, *A*). If the water to be conveyed is relatively clear and experience in the neighborhood indicates that excessive moss or larval conditions will not be present, then full utilization of smooth surfaces may be made. If a smooth surface is to be masked by silt, or moss, insect eggs, and larvae, any added expense for smooth surfaces is hardly warranted as these influences are likely to be effective on smooth and rough surfaces alike and tend to bring them to uniformity (pl. 1, *B*). The basic resulting surfaces, formed by the last two methods, are so dissimilar in characteristics that they require some separate discussion, although both are essentially concrete.

Types I and II cover basic values of n for poured and shot concrete respectively. Variations from these basic values are given in types III to VII inclusive.

I. Poured concrete, basic value, $n=0.014$.—This is the value used for many years by the Bureau of Reclamation for poured-concrete canal linings. It is conservative for ordinary conditions, in that modern methods yield original surfaces at least one or two points lower and hence some acquired roughness has been discounted (pl. 1, *A*). Where the original surface will not change, this value conforms to surfaces as left by smooth-jointed forms or to be roughly troweled. Where alinement is about equal in curves and tangent lengths. The bed to be clean and free from rough deposits. This value also applicable to shot-concrete surfaces, when cleaned and well broomed (pl. 2, *B*) or shot against smooth wood or steel facing (i. e., the wood or steel is on the water side (*57 pl. 1, C*).

II. Shot concrete, basic value, $n=0.017$.—For concrete shot on canal bed or side walls, against a smoothly trimmed earth base or against board or steel backing (opposite from the water side) this value is applicable if the resulting surface is not subjected to any smoothing treatment and is to carry clear water. The surface, without heavy aquatic growth, is distinctly rough, with individual sand and small pebbles clearly outstanding. The hand cannot be slid over such a surface without being scratched. Beside this rough prickly local surface the secondary roughness is developed by undulations an inch or more between the planes of summits and valleys (pl. 3, *C*). Where maximum capacity is not of moment or where the capacity of the rough surface still is sufficient for the maximum supply of water, this type of concrete is excellent in that it has great density and hence is very watertight. Usually the rebound strips the loose pebbles of their cement coatings and thus results in a concrete rich in cement. This is particularly true at the top of canal side walls, the mixture getting leaner as the bottom is reached. The rebound should not be allowed to set where it falls as it contributes to an excessive roughness and has little value in cementing properties and strength. If left in the canal, such a bottom is easily pierced with a steel rod in the hand; hence easily breaks up and loses the desired watertight properties.

$n=0.017$ is also applicable to very roughly coated poured linings with uneven expansion joints.

Conditions under which departures from these basic values of n can be anticipated are described below. In hydraulic research work with models, similarity of surface may require approach to exceedingly smooth surfaces and computations in terms of very low values of n. For such work the reader is referred to Stile's tables (*60*) which begin at $n=0.006$.

III. $n=0.012$ for hard, poured concrete of the highest grade of material and workmanship and exceptionally good conditions. Quite generally used for design from 1900 to 1915. Since then 0.014 has been used under the same conditions. The surface of the lining to be as smooth to the hand as a troweled sidewalk. The expansion joints, if any, to be so well masked that they practically fulfill the same conditions. The climate and water to be such that moss, mud, or insect larvae do not accumulate to any great extent. These items can usually be checked by inspections of other canals in the neighborhood. The water should be free of shifting material. The alinement should be of long tangents joined by smooth gentle curves. The channel must be of true dimensions and laid to uniform grades. This value, or even one point lower ($n=0.011$) is more readily obtained in a semicircular or parabolic form than in the trapezoidal form. This may be in small part due to the added hydraulic efficiency of the curved shape, but is more probably due to the use of oiled-steel forms in obtaining these shapes. This value should seldom be used in design. It will often be attained in the original surface under modern (1937) methods but will too often be radically changed under operative conditions. Shot concrete has been surface-treated so well that this value has been attained, but it is probably at least one point (0.001) too low for design use.

IV. $n=0.013$ for construction as in type III but with curves as in the usual mountain canyon. Same construction and alignment as in type III but with small amount of sand or debris in the water. Construction as in type I but with very favorable alignment or for water that carries a very small amount of colloidal silts that will form a thin slick coating on an original surface of slight roughness. This value of n can be used for design in the conveyance of clear water where excellent concrete surfaces can be assured under specifications calling for premium work and where conditions are such that the original surface is reasonably certain to remain the long-time operative surface. Shot concrete that has been given the best of smoothing treatment with trowel or struck with a steel blade, so that it rates equally with poured concrete of best workmanship, can then be considered under the specifications introducing this paragraph. One large-area aqueduct floor was brought to an even surface with a metal screed, then smoothed with a wooden float and finally polished with a metal hand trowel. The result would justify 0.013 for design and probably rate 0.012 or less as an initial value.

V. $n=0.015$ for construction as in type I but with sharp curves and clean bottom or moderate curves and much debris on the bottom but clean-cut sides. This is about the value to use for the conveyance of muddy waters of streams like Rio Grande and the Colorado (of the far West) in either poured or shot concrete. That is, both smooth and rough concrete as originally laid down, are likely to arrive at a fairly common surface. A slick mud coating with mossy reinforcement sometimes takes on the characteristics of soft suede leather and can be peeled off the surface in flakes as large as the hand. This toughness produces a surface that has fair capacity and is not easily eroded. This value of n should be used in design for broomed shot concrete with either clear or muddy water although lesser values may be attained. Smooth concrete that is seasonally roughened by insect larvae or algal growths takes a value of $n=0.015$ or higher. Consideration should be given to the concurrent seasonal water supply. If it is always inadequate to meet the canal capacity during the same months that bring the seasonal reduction in that capacity, the canal may be designed with a lower value of n, say 0.014, and still be large enough to carry midsummer flows.

VI. $n=0.016$ for lining made with rough board forms conveying clear water with small amounts of detritus. For well-made concrete in deep cuts leading to or from tunnels and subject to heavy contributions of rocky detritus from steep-cut banks. For old linings that have been improved with a thin cement mortar coat, since this coat is quite liable to spall off, especially in a country of cold winters. For smooth or rough concrete if conveying waters that develop moderate coatings of lime ($CaCo_3$).

Lining of original type I eventually takes about this value if subject to reasonably heavy moss growth or large amounts of cobble detritus. Likewise, maximum values of n due to larval growth take about this value.

VII. $n=0.018$ for very rough concrete with sharp curves and deposits of gravel and moss. A broken gradient, irregular cross section, and the like, contribute to this high value of n.

Where experiments show higher values of n than those given above for concrete linings, the conditions are such, as a rule, that the containing material has lost its identity as concrete, and thick coatings of sand, accumulations of moss, or deposits of sand change the general classification of the channel.

VALUES OF n FOR MISCELLANEOUS CHANNEL MATERIALS

1. Cobble-bottom canals. The typical clean-washed cobblestone canal is so common near the mouths of canyons that it should be recognized as in a distinct category. Where the cobbles are graded in size and well packed the value of n is about 0.027, increasing as the larger rocks predominate and the lack of graded sizes prevents packing, and reaching 0.040 for large boulders and heavy sand. The value of n is reduced as silt masks the cobbles and the usual earth canal approached.

2. Wood-plank linings, $n=0.016$ and up. Since these linings are in contact with the ground, they allow grass and weeds to become well rooted in the cracks. Likewise, they are generally uneven with patches of mud and moss (pl. 10, *B*).

3. Rubble and concrete combination, $n=0.015$ and up. Where clear water is carried in a smooth bottom and well-plastered sides the lower value will hold. If the sides are unplastered, $n=0.017$ is a minimum value to use. For fair rock bottom and shot-concrete sides, $n=0.017$ and up. For rubble channels, both sides and bottom roughly covered with cement mortar a value of $n=0.025$ may be reached.

4. Uniform rubble, or concrete sides only and natural-channel bed (pl. 12, *B*), $n=0.018$ and up. For canals and rectified river channels. Concrete or relatively smooth rock bottom and wood-plank sides, $n=0.017$ and up. The lower values will apply if the plank lining is held away from the side wall (pl. 8, *C*).

5. Rock-cut canals, $n=0.035$ or less. The higher value holds for untouched rock cuts, based on the "paper" cross section. Overbreak makes the channel larger with a net slower velocity than for the paper section. Probably the actual value, based on the intricacies of the broken surface, runs about $n=0.040$, but it is hardly feasible to anticipate the extent and formation in the overbreak, hence use of the design section and the lower value of n is better. Where the cut is smoothed up with shot concrete, of course the values of n drop rapidly. Usually such a surface is very undulating; also, such a canal is generally given a high velocity, as the work is expensive. Thus mere surface appearance should not lead to the selection of a too-definitely exact value of n. The local areas may justify 0.017 but the undulations require a value of about 0.025.

More discussion of flow in rock-cut channels will be found under the following authorities: Ramser (*50*, *51*), Sherman (*59*), Mugnier (*44*), DeLacy (*12*), and Newman (*45*).

VARIATIONS OF n IN THE SAME CANAL

It is well known that the same channel does not necessarily have the same value of n throughout the season. Aquatic growths, especially moss, may so change the value of n from early spring to the middle of summer that the channel may carry but 75 percent of its rated capacity for the same depth of water in the channel. The author made a series of current-meter measurements on a prominent lined canal near Yakima, Wash., during July. The canal conveyed more than 70 second-feet when clean in the spring, but in July it carried but 62 second-feet, although bankfull. The velocity was retarded by moss accumulations. As the total capacity of the canal is needed throughout the season, it would have been better to design this concrete channel for a value of n about 0.017 for moss and sharp bends than to use a value of 0.012 as was done. A high velocity would have resulted during the months when the canal was clean, but this would not have injured the concrete. Insect larvae have much the same seasonal effect.

Occasional comments are made to the effect that the value of n changes with the depth of water in the same channel. If this is true the data in figure 5 indicate the change is very slight and quite negligible. While it is sometimes shown by a series of measurements in the same channel, the author believes the change is largely due to differences in surface between bottom and sides of the channel, especially in a lined canal.[22] Almost invariably the bottom is rougher than the sides, especially in the older canals. This is true of both poured and shot concrete. In fact the bottom should be considered first when increase of the capacity of a concrete-lined canal is attempted. If the water is free of sand, a hard smooth inner lining of rich cement-and-sand mortar will materially increase the canal capacity. Many otherwise good linings have a bottom corresponding to a value of n of 0.017 or more while the sides rate less than 0.014. Thin inner linings should be well bonded with the old surface, otherwise they will scale and spall off so that the surface may become rougher than before the improvement was attempted.

CONCLUSIONS

A careful study of the data in this bulletin and the extensive literature bearing on the capacity of canals, together with first-hand observations of such channels with an especially critical interest in their capacity features over a period of nearly 30 years, warrant the following conclusions.

(1) The Kutter (Ganguillet-Kutter) formula is applicable to the design and operation of open artificial channels of sizes in general use. It is favored over all others in most countries where irrigation canals are common, and is almost universally used for canals in hydroelectric practice.

(2) The Manning formula can be used with the same values of n' as for n in the Kutter formula over a range between about 0.012 and 0.020 or, in other words, through the zone most used for canals in concrete, wood, or metal.

(3) Caution should be used in design by the Manning formula outside the range given in (2) above, if the designer is thinking in terms of a value of n in the Kutter formula. (Over a large range of R and S, an equivalent value of n' for n may be found in published tables (*34*).)

(4) Manning's formula is much the easier to solve as a problem in simple mathematics and also lends itself for application where any of the usual functions S, V, R, and n, are to be used in connection with some other phase of hydraulics, such as the solution of backwater curve problem.

(5) Shortcomings are recognized in the Kutter formula but their influence is only of moment under conditions seldom found in canal practice. (The new All-American canal, in southern California may bring out such influences, as R and V are both quite high and the slope S is very gentle, thus allowing the mooted slope factor in Kutter to become effective.)

(6) The factor n must include all the influences which tend to retard velocity. The principal influences are undoubtedly (a) rubbing friction between the water and the containing channels, and (b) vegetal growth extending into the main body of the water. The lack of carrying capacity in many channels is probably due to the fact that the first-cited influence was the only one considered. Of secondary importance, but nevertheless deserving of careful consideration in about the order named, are the following: (c) Angles and

[22] DARR, A. L. See footnote 13.

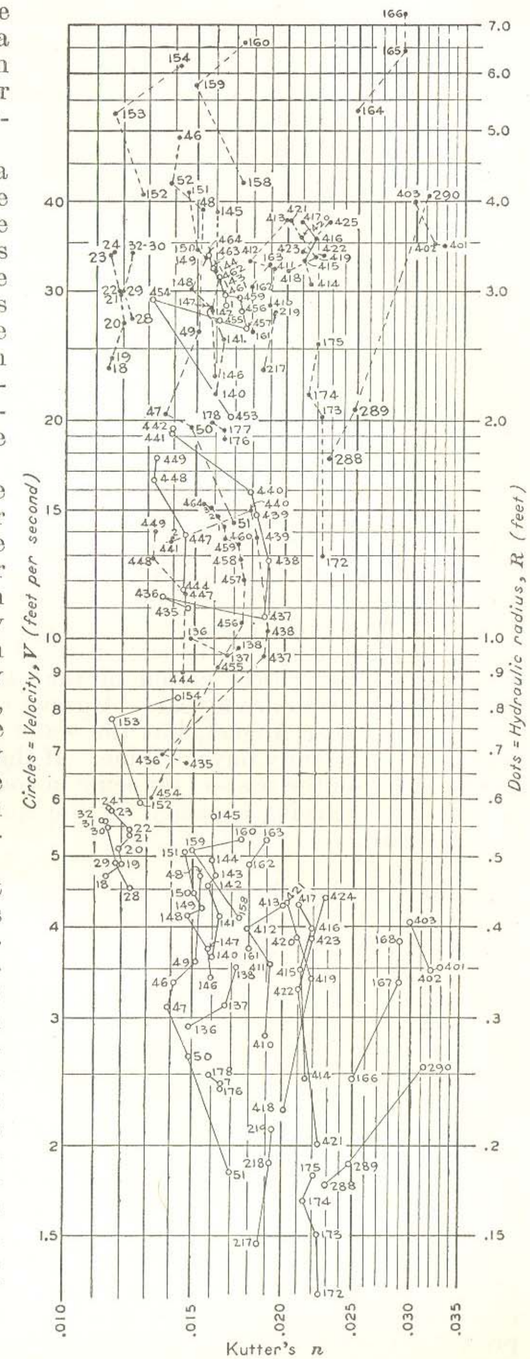

FIGURE 5.—Diagram showing variation of n with varying velocities (open circles) and with varying hydraulic radii (dots) as the discharge is varied in the same reach of the same channels.

sharp curves in the alinement. (d) Influences that tend to accentuate sinuous currents. The concrete lining in a rough-rock cut may feel quite smooth and yet be so undulating as to cause heavy cross currents which retard velocity. All projections and irregularities in the bank of a canal disturb the flow in addition to exposing a large area to rubbing friction. (e) Sand and gravel cause heavy loss in velocity when allowed to enter and accumulate in shifting patches on a lined canal bed. Fine sand drifts downstream in deep, irregular pockets and may entirely change the character of the bottom of a smoothly lined canal. On the other hand, a water laden with fine silt flows more freely after the silt has deposited in a slick coat over minor irregularities than in a new, though clean, canal. A canal carrying such a water may be designed for a far higher velocity through the same kind of soil than would be the case if the water were clear. It is necessary only to run low heads in the new canal until a thick waxy deposit has been placed on the canal bed, after which the velocity may be nearly doubled over that which would have scoured the material in which the canal was originally excavated. (f) The prevailing wind direction may be given some consideration. A study of vertical velocity curves shows a marked change in form with change in wind condition. A downstream wind aids the flow of surface water to the extent that it has the maximum velocity in the vertical, while an upstream wind so shapes the velocity curve that the surface velocity is as slow as that near the bottom.

(7) A value of n must be chosen which will apply to the canal in question at the critical period of the season. For instance, most canals are cleaned once a year. A growth of moss may become very heavy by July or August, but the water supply may be much less than during the early days of June. If the canal is designed to carry its peak load on the basis of its being in good condition, there will still be sufficient carrying capacity for the smaller discharge when moss has appeared.

(8) In the design of earth channels having a trapezoidal form when constructed, the computations should be based on the expectation that the canal will take an elliptical form within a short time and thereafter maintain this shape unless altered artificially.

(9) Capacity of old rough concrete canals can be materially increased at a fraction of the capital charge for the original construction.

(10) Shot concrete should be surface-treated to secure high capacity. If capacity is sufficient anyway then water tightness is better assured without surface treatment, as this includes a tendency toward more porous concrete.

(11) Initial high capacity due to smooth surfaces is not a permanent feature where muddy waters predominate. Such water increases the capacity of a very rough channel and decreases the capacity of a very smooth one.

(12) Determinations of the values of n in experimental research work should not be conducted on the basis of the surface slope. The energy slope is the effective quantity and agrees with the surface slope only for uniform flow, and uniform flow cannot be assumed to be present. Generally it is not. If cross-sectional areas are developed at the ends of a test reach, no matter how uniform the flow looks or should be, it is generally found that these areas are unlike. Hence the velocities are unlike and have different investments of velocity head in their total energy contents.

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